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COMPUTER DESIGN AND ANALYSIS OF SIMPLE SPAN PRESTRESSED CONCRETE HIGHWAY BRIDGE GIRDERS

A THESIS

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COMPUTER DESIGN AND ANALYSIS OF SIMPLE SPAN

PRESTRESSED CONCRETE HIGHWAY BRIDGE GIRDERS

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NOTATIONS

Cross Section:

A	= gross area of precast concrete section
A_s	= total area of prestressing steel
BB(BT)	= width of bottom (top) flange
BW	= web width
BS	= slab width
BE	= effective slab width (= BS $\cdot \frac{E_{cs}}{E_{cb}}$)
CB(CT)	= depth of bottom (top) flange taper
C_b, C_t, C_m	= dimensionless constants where $C_b = \frac{\bar{y}_b}{H}$, $C_t = \frac{\bar{y}_t}{H}$, and $C_m = \frac{y_m}{H}$
cgc	= center of gravity of precast section
cgs	= center of gravity of prestressing steel
GS	= precast inset into slab
H	= depth of precast section
HW	= web depth
I	= moment of inertia of precast section
k_b, k_t	= dimensionless constants where $k_b = \frac{Z_b}{Z_{bc}}$ and $k_t = \frac{Z_t}{Z_{tc}}$
K_c	= kern distance of bottom fiber (= Z_b/A)
TS	= slab thickness
$\bar{y}_b$ ($\bar{y}_t$)	= distance from bottom (top) fiber to neutral axis of precast section

y_{bc} (y_{tc}) = distance from bottom (top) fiber to neutral axis of composite section

y_e = y_b required by $e_s + y_s$

y_u = required value of y_b for a section controlled by ultimate flexural strength

y_m = value of y_b for a section with both the top and bottom section moduli at their minimum allowable values

y_s = distance from bottom fiber to cgs

Z_b (Z_t) = bottom (top) section modulus of precast section

Z_{bc} (Z_{tc}) = bottom (top) section modulus of composite section

Load and Prestress:

α = fraction of the beam weight moment affecting the required section moduli

β = fraction of the beam weight moment counteracting the effect of the prestressing force at transfer

e_s = eccentricity of prestressing steel at midspan

e_u (e_ℓ) = upper (lower) limiting value of e_s

F_o = prestress force at transfer

F_u (F_ℓ) = maximum (minimum) F_o required for prestressing steel at $e_u(e_\ell)$

L = span length

S = length of horizontal segment of harped tendon

M_G = midspan moment due to weight of beam

M_s = midspan moment due to superimposed loads both dead and live

M_u = midspan moment due to total ultimate loads

M_c = midspan moment due to weight of slab

η = fraction of initial prestressing force remaining after losses

W = weight of beam per foot

Concrete and Steel Stresses:

f'_c = 28 day concrete strength

f'_{ci} = at transfer strength of concrete

f'_s = ultimate strength of prestressing steel

FCP1 (FTP1) = allowable compressive (tensile) stress at prestressing stage

FCP2 (FTP2) = allowable compressive (tensile) stress at working load after losses

f_a = average concrete stress due to prestressing force only

f_{au} ($f_{a\ell}$) = maximum (minimum) average concrete stress due to F_u (F_ℓ) at e_u (e_ℓ)

f_F^b (f_F^t) = concrete stress on bottom (top) fiber due to F_o

f_{Fu}^t ($f_{F\ell}^t$) = top fiber stress due to F_u (F_ℓ) at e_u (e_ℓ)

f_{Fu}^b ($f_{F\ell}^b$) = bottom fiber stress due to F_u (F_ℓ) at e_u (e_ℓ)

$f_{\ell c}^b$ ($f_{\ell c}^t$) = the stress in the bottom (top) fiber of the precast beam due to the live load on the composite section

f_ℓ^{bs} (f_ℓ^{ts}) = the stress in the bottom (top) fiber of the slab due to the live load on the composite section

f_{auc} = maximum allowable average concrete stress in the precast beam due to F_u at e_u

E_{cb} (E_{cs}) = modulus of elasticity of the concrete beam (slab)

Ultimate Strength:

b = effective width of area acted upon by stress block may be "BE" for composite sections or "BT" for flanged sections or "BB" for rectangular sections

C1 = compressive force developed if stress block were to cover the entire slab

$= .85\ f'_c \cdot TS \cdot BE$

C2 = compressive force developed if stress block were to cover non-tapered portion of flange only

$= .85\ f'_c\ (TS - GS) \cdot BT$

C3 = compressive force developed if stress block were to cover tapered portion of flange only

$= .85\ f'_c \cdot (BT + BW) \cdot CT/2.0$

d = distance from extreme compressive fiber to centroid of prestressing force

$= H - y_s + (TS - GS)$

M'_u = ultimate flexural capacity

P = ratio of prestressing steel

$= A_s/bd$

f_{sy} = nominal yield strength of prestressing steel

f_{se} = effective prestress after losses

f_{su} = calculated stress in prestressing steel at ultimate load

SUMMARY

A computer program has been developed to design and analyze simple span prestressed concrete highway bridge girders because of the time consuming and tedious hand computations often involved in such a design.

An iterative working stress design procedure is used based on just meeting all stress requirements during prestressing and under full working loads. The section is checked for violation of ultimate strength requirements and redesigned if necessary. Design computations are based on the gross section properties while analysis computations are based on the net/transformed section properties.

The computer program is equipped to handle I, T, and rectangular concrete sections with either composite or non-composite action. Loading input may include combinations of uniform, uniform segment, or concentrated static loading combined with any one of the AASHO standard truck or lane loadings. Maximum and minimum section dimensions and certain dimension ratios are provided as user input options. Other input options include: tendon geometry, bonding and tensioning specifications, stress allowables, modulus of elasticity and specific weight of concrete, and a creep factor for deflection computations.

The computer-aided design technique described herein has been found to provide a means of reducing design time and costs while producing an economical and efficient prestressed concrete section. The computer design also presents the designer with a valuable tool for making studies

as to the effect of parameters such as section dimensions, span length, concrete strength, and tendon geometry on the economy of the girder.

CHAPTER I

INTRODUCTION

The purpose of this thesis is to develop a philosophical and logical method for the design and analysis of prestressed concrete highway girders which may be applied to a programmed design technique.

At present, prestressed concrete highway girders are designed by selecting a standard AASHO-PCI section and checking it for adequacy as it passes through several important loading stages. This method, although easy to apply, has the disadvantage of providing, in many cases, a girder which is understressed in say the top fiber while marginally meeting stress requirements in the bottom fiber, or one which is well understressed simply because it was the next larger standard section from an inadequate one. An acceptable design occurs when the section just meets the allowable stress requirements both during the prestressing stages (at transfer) and under full service working loads (final). While this condition may be desirable from an economic and theoretical standpoint, the laborious and often tedious computations required for such a design cannot be justified for manual computations.

It is for this reason that a computer program has been developed to handle the involved computations required to produce a section based on the minimum allowable stress requirements. The computer aided design technique described herein has been found to provide a means of reducing design time and costs while producing an economical and efficient prestressed concrete section.

There are, of course, programs written for the analysis of prestressed concrete beams. The Portland Cement Association has developed a series of programs to analyze and design simple spanned, composite highway and railway bridges (1).* The disadvantage to the "design" procedure used in the PCA program is that all dimensions of the precast section must be input while the program determines what combination of prestress force and eccentricity will not overstress the section. This method certainly yields an acceptable design. However, since the precast section is input, the so called "design" is very limited.

Stubbs (2) has proposed that design begin with a concrete section, that some prestress force be assumed, and the beam capacity be computed. If the beam will not carry the imposed loads, more prestressing should be added and the beam capacity recomputed. If the beam will not carry the imposed loads, more prestressing should be added, etc. This method not only has the disadvantage of having to input the concrete section dimensions, but there is no guarantee that increasing the prestress force will result in an acceptable design.

When the concrete section must be input by the user it is difficult to say that the program actually designs the beam. A true design program should rely only on basic input information such as loadings, span length, tendon shape, geometric constraints, etc., and should produce an acceptable design based on this input information. The computer pro-

*Numbers in parentheses refer to references in the bibliography.

gram developed as part of this thesis affords the complete design of a simple spanned prestressed concrete bridge girder based on only the minimum essential design information. The user of the program need not input any information regarding the section dimensions, prestress force, or tendon eccentricity. All these parameters are computed by the program.

Since there exist certain limitations as to space available, precasting forms, and practical dimensions, a "free" design procedure may produce an impractical section. It is for this reason that certain maximum and minimum dimension constraints as well as certain maximum and minimum width to thickness ratios are provided as optional input to the user. The specification of these options insures the design to be within a certain range of practical dimension limitations.

The program has been developed to examine each applied loading and generate a maximum moment envelope from these specified loadings. The design method employed is based on a method described by Wang (3,4) which yields a balanced prestressed concrete section with a minimum bottom section modulus and a minimum top section modulus (if permitted by tendon requirements). This method produces a section exhibiting initial and final top and bottom fiber stresses at their maximum allowable values. It not only produces a balanced design, but also a design which satisfies all user imposed constraints.

The program's basic purpose is that of design but it may also be used to check any input section for a series of loading conditions. The check mode of the program outputs the maximum envelope of moments, stresses, and deflections for a given girder section, prestress force and tendon eccentricity based on the net/transformed section properties.

The program handles both composite and non-composite sections and is applicable to I, T, and rectangular shaped beams. The user may specify either a parabolic tendon, a straight tendon, or a harped tendon with a maximum of two symmetrical hold down points. The tendon may be either pretensioned or post-tensioned, bonded or unbonded. Loadings include any number and combination of uniform, uniform segment, and concentrated loads combined with any one of the AASHO standard truck or equivalent lane loadings. Since the program is intended for the design of simple span highway bridge girders, the AASHO code specifications are used throughout, but the program input is flexible enough to override the AASHO specifications if the need arises.

The usefulness of the program is demonstrated by the example problems shown in Chapter II. It affords the user a quick, efficient, and inexpensive means of designing or checking prestressed concrete girders. The designer is not limited to the standard AASHO-PCI prestressed concrete sections which means he may now design a more economical and efficient section. The designer is also presented a tool for making studies as to the effect of parameters such as section dimensions, span length, concrete strength, etc. on the economy of the beam.

CHAPTER II

RESULTS

The advantages of a programmed design as opposed to traditional manual computations are many. The programmed design frees the engineer from time consuming, tedious, and repetitive computations so that he may direct his energies toward more meaningful ends such as optimization studies. The computer program insures that computations are correct and exact, eliminating the risk of computational error. The usefulness of a computer-aided design does not end with computational exactness and coordination, rather the program becomes a valuable tool to aid in the engineering decision making process.

It is the intent of this chapter to demonstrate the capabilities, usefulness, and efficiency of the proposed programmed design technique.

Girder Design

The primary purpose in developing the proposed prestressed concrete design program was to make it applicable to the design of simple-span highway bridge girders. The first example problem, therefore, will be the design of such a girder.

Suppose one desired to design a simple-span prestressed concrete girder bridging a 75 foot span. Each girder is to support one-half of the lane load produced by a standard HS20-44 truck loading and act in composite action with a 72 inch wide, 8 inch thick slab. The cast-in-place slab is to be supported by scaffolding during construction. The prestressing tendon is to be post-tensioned, bonded by grouting, and parabolic in shape.

Inputing this information into the program, the following design is generated:

Depth	=	45.0 in.
Top Flange Width	=	39.1 in.
Top Flange Thickness	=	5.0 in.
Web Thickness	=	6.0 in.
Bottom Flange Thickness	=	5.0 in.
Bottom Flange Width	=	24.9 in.

The initial prestress force is determined to be 547.8 kips requiring 2.90 square inches of steel. Besides giving the required prestress force and area of steel, the program outputs a band of tendon profile limits and the computed tendon profile lying between the profile limits. Table 1 gives the tendon profile data which were output for this design. The deflected shape of the beam is output for each loading condition specified by the user. For the beam designed by the program for this example, an upward camber at midspan of 0.45 inch was computed due to prestressing and a maximum downward deflection of 0.29 inch at midspan was computed under full service loads.

Besides outputing the information required to physically construct the girder, the program also prints out all the precast beam and composite section properties, the maximum midspan moments, and the maximum moment envelope, the ultimate flexual capacity, the allowable and the actual stresses at transfer and at full working load in both the top and bottom fibers, the maximum stress in the top fiber of the slab, and the initial stress in the prestressing tendon.

Table 1. Tendon Profile Data

Distance from Left Support (Feet)	Upper Limiting Eccentricity (Inches)	Lower Limiting Eccentricity (Inches)	Tendon Profile (Inches)
0.00	- 4.60	7.95	0.00
7.50	1.91	10.07	4.98
15.00	6.95	11.72	8.86
22.50	10.60	12.90	11.63
30.00	13.00	13.61	13.29
37.50	13.84	13.84	13.84
45.00	13.00	13.61	13.29
52.50	10.60	12.90	11.63
60.00	6.95	11.72	8.86
67.50	1.91	10.07	4.98
75.00	-4.60	7.95	0.00

Computerized Design vs. AASHO Standardized Section

An advantage of the programmed design technique proposed herein is the automatic generation of an economical and efficient prestressed concrete girder. To exemplify this advantage over standard manual design techniques, a programmed design will be compared with a manual design example presented by Gaylord and Gaylord (5).

In this example, the design is required for a composite highway bridge girder to resist an HS20-44 live load and a 100 plf dead load over a 70 foot span. The design example assumes a Type III AASHO-PCI beam and determines the prestress force and concrete strength necessary for a satisfactory design. The same loadings, stress allowables concrete strength, slab dimensions, etc., were input into the program and, as shown by Figure 1 and Table 2, a smaller section was generated. The smaller section is possible because the computer design produces a section which exactly meets all stress allowables.

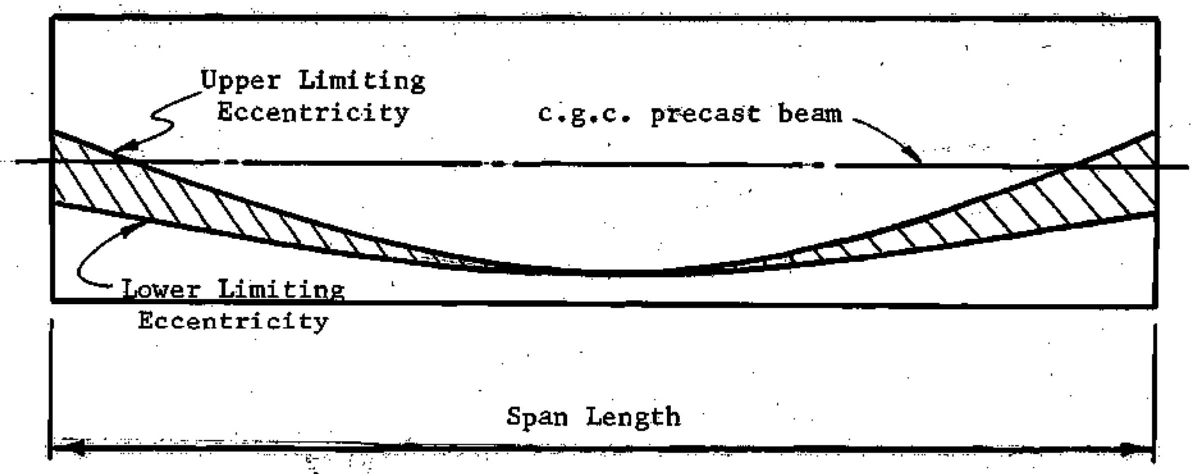


Figure 1. Band of Acceptable Tendon Profiles

Table 2. Computer Designed Section vs. Standard AASHO Section

	Computer Designed Section	AASHO Type III Section
Concrete Area	412 in.^{2}	560 in.^{2}
Moment of Inertia	70,554 in.^{2}	125,390 in.^{2}
Initial Prestress Force	654 kips	739 kips
Area of Prestressing Steel	3.74 in.^{2}	7.28 in.^{2}
Area of Non-Prestressing Steel	0.00 in.^{2}	0.48 in.^{2}

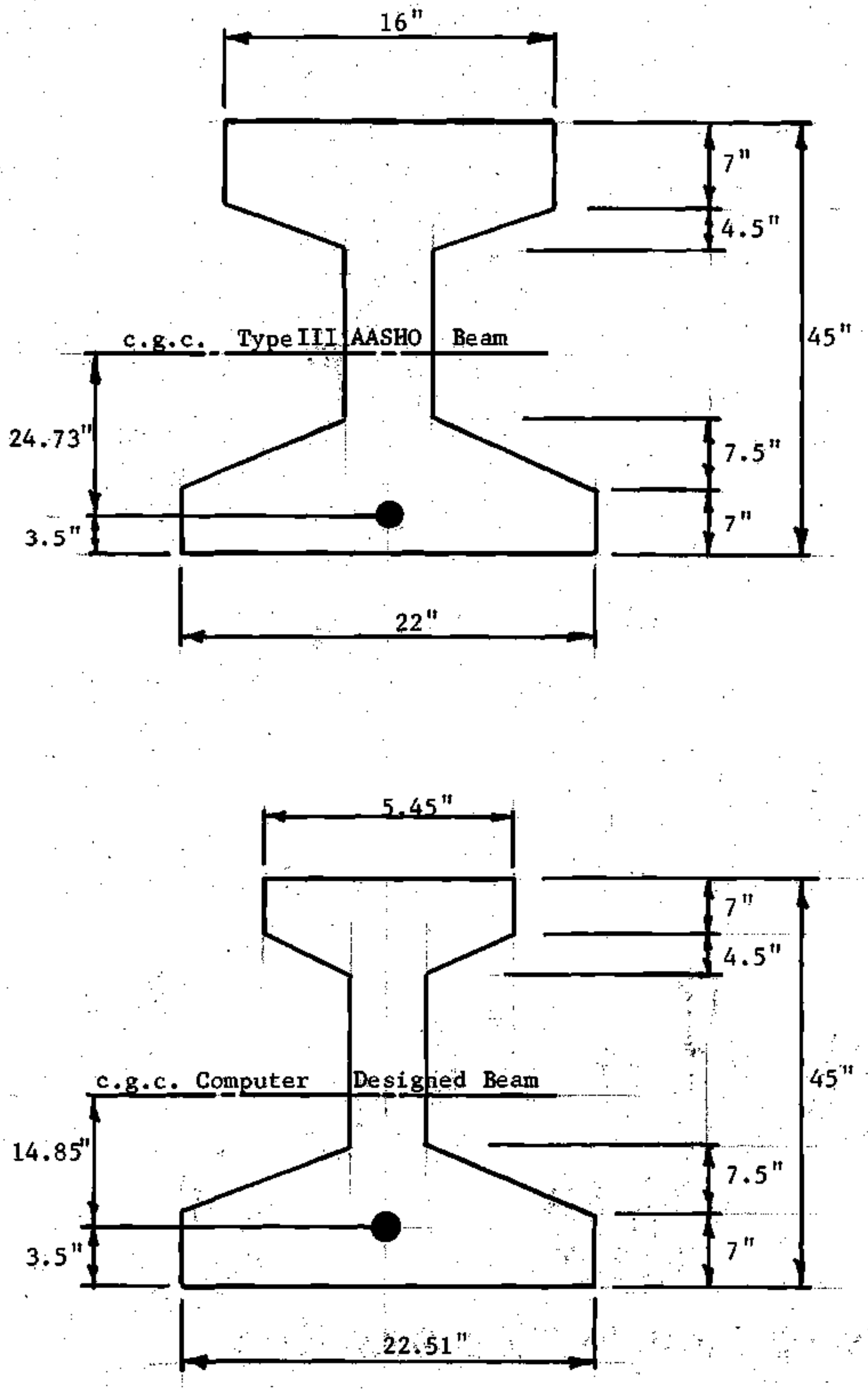


Figure 2. Computer Designed Section vs. Standard AASHO Section

Optimum Depth

Usually the depth of a girder is determined by the maximum depth permitted before violating a predicted physical limitation such as underpass clearance requirements. Normally it is desired to make the girder as deep as possible in order to place the most material as far as possible from the neutral axis. In the case where the girder depth is not limited by any physical restraints, beam efficiency and economy increase with girder depth until a point is reached when further increases in depth begin to decrease girder economy. The decrease in economy occurs because, after a point, the minimum allowable section dimensions begin to control the girder size rather than the stress requirements. Without the aid of a programmed design, it would be a long laborious procedure to determine the optimum section depth for a set of given conditions. But by merely inputing several sets of data, each the same except for girder depth, a family of designs is generated in which the required area of concrete decreases with depth up to a point, after which it begins to increase. At this transition, the optimum depth is achieved with respect to the required concrete area.

To demonstrate this situation, an I-shaped, simple-span composite section with a post-tensioned, bonded tendon and spanning 75 feet was designed. Minimum dimension constraints were specified as two inches for the flange thicknesses and four inches for the flange widths and the web thickness. The minimum flange thickness to width ratio was specified as 0.075 for both flanges and the minimum web thickness to total depth ratio was specified as 0.10. The top flange taper was fixed at 4.0 inches and the bottom flange taper was held at 6.5 inches. A HS20-44 standard truck load was distributed over two girders and a uniform load of 0.050 kip per foot was included. Computer designs were generated for six girders, the first with a fixed depth of 30.0 inches and five more with fixed depths, respectively, of 35.0, 40.0, 45.0, 50.0, and 55.0 inches. The required area of concrete for each design was as follows:

Depth (in.)	Area of Concrete (in.^{2})
30	550.3
35	462.0
40	336.3
45	298.4
50	292.5
55	313.9

The 50 inch depth requires the least area of concrete; therefore, in the case where girder depth is unrestrained, a depth in the range of 45 inches to 55 inches would require the least volume of concrete.

Optimum Span Length

The span length for overpass construction is usually determined by the physical requirements at hand but when a structure must bridge a relatively long distance it becomes important to determine the best combination of span length and pier requirements. For example, if a 1000 foot distance had to be spanned and assuming that only simple span construction is permitted, would it be more economical to use ten 100 foot spans with 9 piers (it is assumed that the span is supported by something other than piers on each end) or twenty 50 foot spans with 18 piers? Obviously the answer to this question is dependent upon a number of factors, but a simplified approach may be to determine which combination requires the least area of concrete. A further simplification may be to assume that a standard pier size be taken independent of the span length. By making such assumptions, it would be an easy task to determine the cost of a number of combinations of span length and pier numbers if a girder design were readily available for each span length. This is where a programmed design becomes useful. It allows a number of designs to be made, each differing slightly from one another (in this case the span length varies while all other parameters remain constant) and comparisons made to determine the best design.

To demonstrate, a hypothetical case will be considered. Suppose it was required to know which combination of span length and number of piers would require the least volume of concrete to cross a 300 foot portion of swamp land. The volume of a single standard pier grouping has been determined to be 42 cubic yards. The bridge is to be designed for a HS20-44 standard truck load and eight girders will support four lanes of traffic. By inputing this information together with the slab and tendon requirements and the requirement of equal span length, the program would generate a set of data as shown in Table 3.

Table 3. Optimum Span Length Determination

Span Length (Ft)	No. Spans	No. Piers	Total Volume of Girders (Yds^3)	Total Volume of Piers (Yds^3)	Total Volume (Yds^3)
37.5	8	7	109	294	403
50.0	6	5	170	210	380
60.0	5	4	181	168	349
75.0	4	3	262	126	388
100.0	3	2	422	84	506
150.0	2	1	664	42	706
300.0	1	0	1820	0	1820

As may be seen from Table 3, the least volume of concrete is required for a 60 foot span with four pier groups. This determination was made utilizing several simplifying assumptions but a similar determination, taking into account pier volume dependent upon span length, area of prestressing steel, cost of labor and materials, etc., may also be performed utilizing the design capabilities of a computer program for the girder design.

Section Types

It is seldom that the engineer can afford the time to compare an I, a T, and a rectangular section to determine which best suits the requirements at hand. Usually he depends upon experience and common sense to choose a section and makes a design based on this decision. But with the aid of a design program, it is little trouble to design all three section types and make a realistic comparison based on the program output.

Usually, an I section is best suited to short and intermediate spans with moderate to heavy superimposed loads, while a T section is most efficient for girders permitting relatively large depths and/or light superimposed loads. The rectangular section is employed where simplicity in forming is required or where composite action with a slab is used to duplicate the action of a T section.

As an example, the design was generated for a highway bridge girder spanning 77 feet loaded by an HS20-44 AASHO truck loading and a uniform load of 125 lbs/ft. All design requirements were the same except for the section type. It was found that the I beam required the least cross sectional area (464.7 in.^{2}); the rectangular section fell in between with

833.5 in.^{2}; and the T beam required 1152.1 in.^{2} For the span length required and loadings superimposed, the I beam, based on concrete requirements, would appear to be the most economical section. For different conditions, however, or if the concrete cross sectional area was not the governing factor, a different section type may be found to be more desirable. Another factor, although not considered by the computer program, is the satisfaction of shear requirements. It may be found that the I section, while satisfying moment requirements, will not satisfy shear requirements, and that a rectangular section may be best suited to the total requirements at hand.

After determining the best section type suited to the conditions at hand, the user is in a position to make further investigations into the best number of girders for a single span or the best depth for a single girder, etc.

Gross vs. Net/Transformed Section

As stated in Chapter I, the computer program developed includes both design and checking procedures. The design is based on the gross section while the checking procedures are based on the net/transformed section. It is interesting to note that a girder designed by the program may not necessarily meet the stress requirements if checked by the program. This is because of the difference in section properties computed under the design and check modes of the program. If the section modulus at the bottom fiber is close to the minimum, which it will be when designed by the program, and if the area cored out by the tendons is relatively large, the bottom fiber may be overstressed at transfer initially when checked on the basis of the net cross section. The top fiber will probably be overstressed as well if it was fully employed originally. This situation is easily remedied, however, by maintaining the same section dimensions computed for the gross section design and reducing the prestress force. This solution is possible because at transfer the centroid is raised because of the area removed by the tendon ducts. A larger eccentricity can be realized, which makes for a reduction in the required initial prestress force, FO. At working load, the effective size of the bottom flange is increased due to the transformed steel area. The stress change induced by the superimposed moment is thereby diminished so that the required precompression is less, again reducing the necessary magnitude of FO.

For example, if the girder designed for the first example design in this chapter were checked under the check mode of the program, it would be found to be slightly overstressed in the bottom fiber at transfer. As seen from Table 4, this situation is remedied by decreasing the initial prestress force from 547.8 kips to 540.0 kips.

Table 4. Section Stresses

	Allowable Stresses	Gross Section Design	Net/ Transformed Check	Decreased FO Check
Top Fiber Initially	- 189.7	47.4	55.0	68.9
Bottom Fiber Initially	2400.0	2400.0	2422.9	2378.5
Top Fiber Finally	2000.0	1940.8	1928.2	1940.6
Bottom Fiber Finally	0.0	0.0	112.1	72.5

CHAPTER III

DIRECT DESIGN OF PRESTRESSED CONCRETE SIMPLE SPAN GIRDERS

Design of Non-composite Sections

Usually, the flexural design of a prestressed concrete member begins with a trial section. This section is reviewed for adequacy as it passes through several important loading stages. The dimensions of the section are then revised and adjusted. This is a time consuming operation if manual methods are used and an inefficient one if a computer is used. The method to be presented herein will offer a direct design according to the flexural stress requirements of both prestressing and working load stages. Dimensions of the concrete section and the prestressing force and its eccentricity are directly computed from superimposed loadings and allowable stresses.

The design procedure follows from a method presented by Wang (3,4) and is based on the minimum bottom section modulus, Z_b, and the required distance from the bottom fiber to the neutral axis, y_b. On the basis of the design procedure to be discussed later, any two dimensions of the cross section may be obtained by solving simultaneous moment of inertia and moment-area relationships, but discussion will be limited to solving for either the two unknown flange widths or the two unknown flange thicknesses.

General Tendon Profile

Because the critical section of a simply supported uniform prismatic section is dependent upon the path of the tendon, determination of the minimum section moduli must reflect the shape of the tendon profile. Wang has presented a method to account for some common tendon profiles in determining the minimum required section moduli.

Consider the generalized tendon profile shown in Figure 3. By varying s from zero to L, a parabolic harped (made up of three straight segments) or straight tendon may be specified. For example, s = 0 corresponds to either a parabolic tendon or a harped tendon with a single hold down point at midspan, s = L corresponds to a straight tendon and $0 < s < L$ is a harped tendon with two hold down points (points a and b of Figure 3) at $\pm s/2$ from midspan.

The initial resultant stresses after prestressing (at transfer) are a function of the moment due to the initial prestressing force and its eccentricity plus the counteracting moment due to the weight of the beam. Therefore, the critical section at transfer is at a hold down point and may be expressed as

$$M_{harp} = \frac{WL^2}{8}\left(1 - \frac{s^2}{L^2}\right)$$

where W is the weight per foot of the beam. Now defining $\beta = 1 - \frac{s^2}{L^2}$ and noting that the maximum moment due to the weight of the beam is $M_G = \frac{WL^2}{8}$ one may write $M_{harp} = M_G$.

It should be noted, however, that in the case of a harped tendon with a single hold down point at midspan (s = 0), the previously derived equations do not account for violation of the upper limiting eccentricity immediately to the right and left of midspan. To correct this situation a small value of (in the range of 0.1L to 0.05L) should be specified.

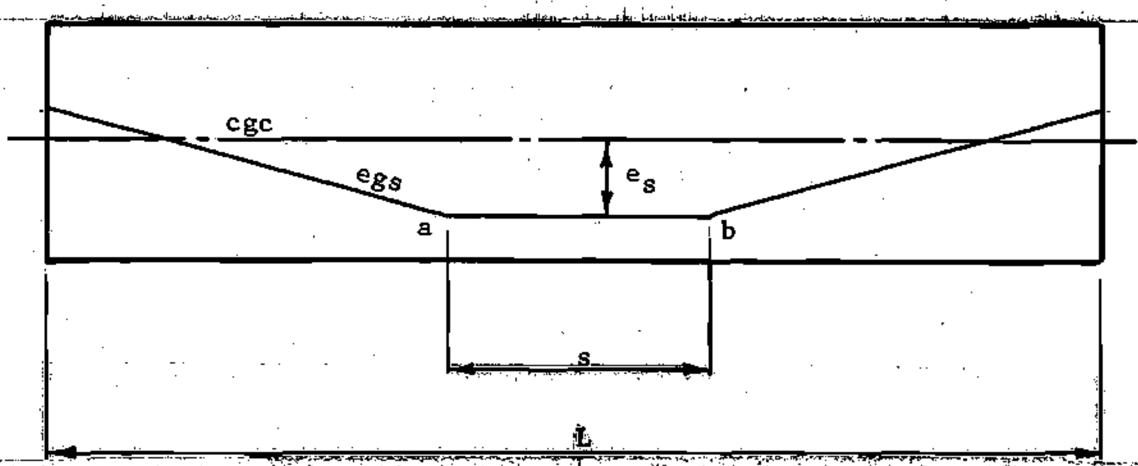


Figure 3. General Tendon Profile

Governing Equations

A simple span prestressed concrete beam is governed by four limiting stress conditions: FCP1, the initial allowable compressive stress; FTP1, the initial allowable tensile stress; FCP2, the final allowable compressive stress; and FTP2, the final allowable tensile stress. Now defining

F_o = initial prestress force at transfer

e_s = midspan eccentricity of prestressing steel

$Z_b(Z_t)$ = bottom (top) section modulus

A = gross area of concrete

M_s = moment due to superimposed dead and live loads, excluding dead weight at time of prestressing

η = fraction of initial prestress force remaining after losses we may express the four limiting stress conditions as inequalities. Initially, at the hold down points, the resultant of the applied stresses must not exceed the compressive stress allowed at the bottom fiber or,

$$FCP1 \geqq \frac{F_o e_s}{Z_b} + \frac{F_o}{A} - \frac{\beta M_G}{Z_b} \tag{1}$$

The resultant stress at the top fiber must not fall below the minimum allowed,

$$FTP1 \leqq \frac{-F_o e_s}{Z_t} + \frac{F_o}{A} + \frac{\beta M_G}{Z_t} \tag{2}$$

At working load, after all prestress losses have taken place, the resultant stress at the top fiber must not exceed the compressive stress allowed,

$$FCP2 \geqq \frac{-\eta F_o e_s}{Z_t} + \frac{\eta F_o}{A} + \frac{M_G}{Z_t} + \frac{M_s}{Z_t} \tag{3}$$

The stress at the bottom fiber must remain above the minimum value permitted,

$$FTP2 \leqq \frac{\eta F_o e_s}{Z_b} + \frac{\eta F_o}{A} - \frac{M_G}{Z_b} - \frac{M_s}{Z_b} \tag{4}$$

In the above inequalities, tensile stresses are considered negative and eccentricities are positive when below the neutral axis.

Section Moduli Requirements

The four governing equations may be used to determine the permissible section moduli. Multiplying (1) by η and (3) by -1 and adding

$$\eta FCP1 - FTP2 \geqq \frac{(1 - \beta\eta)M_G}{Z_b} + \frac{M_s}{Z_b}$$

$$Z_b \geqq \frac{M_s + (1 - \beta\eta)M_G}{\eta FCP1 - FTP2} \tag{5}$$

Similarly, multiplying (2) by $-\eta$ and adding to (4)

$$FCP2 - \eta FTP1 \geqq \frac{(1 - \beta\eta)M_G}{Z_t} + \frac{M_s}{Z_t}$$

$$Z_t \geqq \frac{M_s + (1 - \beta\eta)M_G}{FCP2 - \eta FTP1} \tag{6}$$

Defining $\alpha = 1 - \beta\eta$, the minimum permissible section moduli may be obtained

$$(Z_b)_{min} = \frac{M_s + \alpha M_G}{\eta FCP1 - FTP2}$$

$$(Z_t)_{min} = \frac{M_s + \alpha M_G}{FCP2 - \eta FTP1}$$

Prestressing Force and Its Eccentricity

If Z_b is always greater than or equal to the minimum, then from (5)

$$\frac{M_s + \alpha M_G}{Z_b} \leqq \eta FCP1 - FTP2$$

or

$$\frac{M_s + M_G}{Z_b} - \frac{\eta\beta M_G}{Z_b} \leqq \eta FCP1 - FTP2$$

$$\frac{1}{\eta}\left(FTP2 + \frac{M_s + M_G}{Z_b}\right) \leq FCP1 + \frac{\beta M_G}{Z_b} \tag{7}$$

And similarly for Z_t always greater than or equal to its minimum permissible value

$$FTP1 - \frac{\beta M_G}{Z_t} \leq \frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right) \tag{8}$$

Now defining

f_F^b $= \frac{F_o e_s}{Z_b} + \frac{F_o}{A}$ or the stress in the bottom fiber due to the initial prestress force

f_F^t $= \frac{-F_o e_s}{Z_t} + \frac{F_o}{A}$ or the stress in the top fiber due to the initial prestress force

$f_{F\ell}^b$ $(f_{F\ell}^t)$ = the stress in the bottom (top) fiber due to the prestress force at its lower limiting eccentricity

f_{Fu}^b (f_{Fu}^t) = the stress in the bottom (top) fiber due to the prestress force at its upper limiting eccentricity

As may be seen from Figure 4, f_F^b can be no larger than FCP1 + $\frac{\beta M_G}{Z_b}$ nor any less than $\frac{1}{\eta}\left(\frac{M_s + M_G}{Z_b} + FTP2\right)$. Similarly f_F^t can be no greater than $\frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right)$ nor less than FTP1 - $\frac{\beta M_G}{Z_t}$.

The minimum permissible prestress force at the lower limiting eccentricity is limited by

1) the final tension allowable in the bottom fiber

2) the initial tension allowable in the top fiber or, as seen

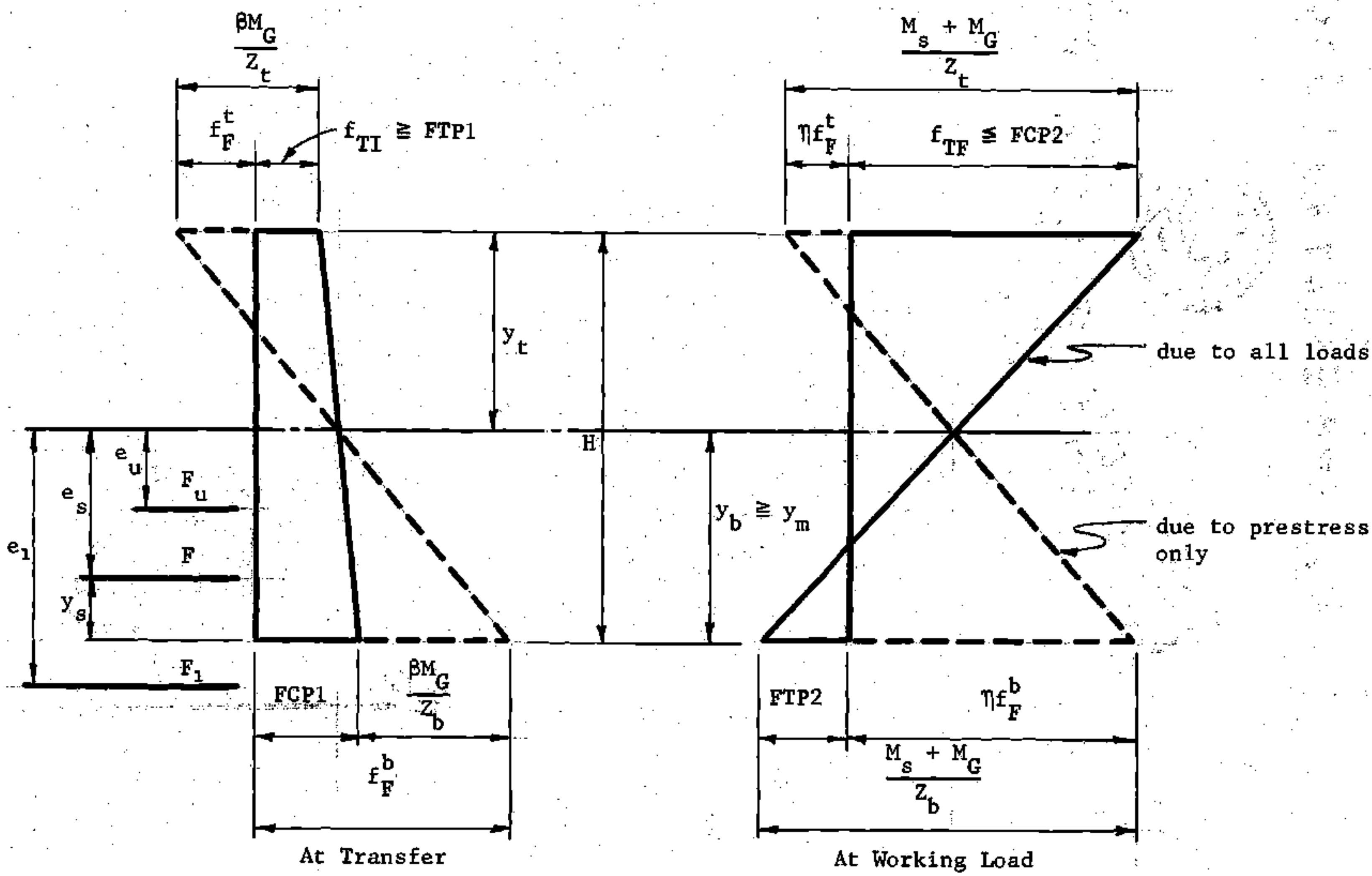


Figure 4. Stress Distribution

from Figure 4,

$$f^b_{F\ell} = \frac{1}{\eta}\left(FTP2 + \frac{M_s + M_G}{Z_b}\right)$$

$$f^t_{F\ell} = FTP1 - \frac{\beta M_G}{Z_t}$$

The maximum permissible prestress force and corresponding eccentricity is limited by

1) the initial compression allowable in the bottom fiber
2) the final compression allowable in the top fiber or,

$$f^b_{F_u} = FCP1 + \frac{\beta M_G}{Z_b}$$

$$f^t_{F_u} = \frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right)$$

We may now recast equations (7) and (8) as bounded quantities,

$$f^b_{F\ell} = \frac{1}{\eta}\left(FTP2 + \frac{M_s + M_G}{Z_b}\right) \leq f^b_F \leq FCP1 + \frac{\beta M_G}{Z_b}\Big) = f^b_{F_u} \tag{9}$$

$$f^t_{F\ell} = FTP1 - \frac{\beta M_G}{Z_t} \leq f^t_F \leq \frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right) = f^t_{F_u} \tag{10}$$

There is an upper and a lower bound to the stress caused by the initial prestress in both the upper and lower fibers. By imposing the condition that the bottom section modulus always be at the minimum value allowed by (5), then (9) becomes an equality thus,

$$f^b_{F\ell} = \frac{1}{\eta}\left(FTP2 + \frac{M_s + M_G}{Z_b}\right) = f^b_F = FCP1 + \frac{\beta M_G}{Z_b} = f^t_{F_u} \tag{11}$$

Two of the four bounds have been fixed by makiig Z_b a minimum. Substituting the general relation $f^b_F = \frac{F_o}{A} + \frac{F_o e_s}{Z_b}$ into (11) yields

$$\frac{F_o}{A} + \frac{F_o e_s}{Z_b} = FCP1 + \frac{\beta M_G}{Z_b} \tag{12}$$

Now making use of the expression for the top kern, $K_c = \frac{Z_b}{A}$, and the average stress on the cross section due to $F_o, f_a = \frac{F_o}{A}$, substitution into (12) yields an expression for e_s

$$e_s = K_c\left(\frac{FCP1}{f_a} - 1\right) + \frac{\beta M_G}{f_a A} \tag{13}$$

for for f_a in terms of e_s,

$$f_a = \frac{K_c FCP1 + \beta M_G}{e_s + K_c} \tag{14}$$

Both (13) and (14) apply only when Z_b is a minimum. A single e_s and f_a may be realized only when both Z_b and Z_t are at their minimums, otherwise a range of e_s and f_a exists.

Consider the case when Z_b is at a minimum but Z_t it something greater than its minimum value, then from (10) f^t_F has two limiting values

minimum: $$f^t_{F\ell} = FTP1 - \frac{\beta M_G}{Z_t} \tag{15}$$

$$\text{maximum:} \qquad f^t_{F_u} = \frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right) \tag{16}$$

Now defining

$$C_b = \frac{y_b}{H} \qquad \text{and} \qquad C_t = \frac{y_t}{H}$$

then

$$y_b = C_b H \qquad \text{and } y_t = C_t H$$

but

$$y_b Z_b = I = y_t Z_t$$

so that,

$$Z_b C_b H = Z_t C_t H$$

or

$$Z_b C_b = Z_t C_t$$

as seen previously,

$$f^b_F = \frac{F_o}{A} + \frac{F_o e_s}{Z_b} \qquad \text{and} \qquad f^t_F = \frac{F_o}{A} - \frac{F_o e_s}{Z_t}$$

substituting $f_a = \frac{F}{A}$ and dividing by C_b and C_t

$$\frac{f^b_F}{C_b} = \frac{f_a}{C_b} + \frac{F_o e_s}{Z_b C_b} \qquad \text{and} \qquad \frac{f^t_F}{C_t} = \frac{f_a}{C_t} - \frac{F_o e_s}{Z_t C_t}$$

adding the two equations and noting that $Z_b C_b = Z_t C_t$,

$$\frac{f^b_F}{C_b} + \frac{f^t_F}{C_t} = \frac{f_a}{C_b} + \frac{f_a}{C_t}$$

multiplying through by $C_b C_t$ and collecting terms

$$f_F^b \, C_t + f_F^t \, C_b = f_a(C_t + C_b)$$

The maximum average concrete stress, f_{au}, produced by the maximum permissible prestressing force at the upper limiting eccentricity may be determined by substituting $f_{Fu}^b = f_{F\ell}^b$ from (11) for f_F^b in the previous equation and f_{Fu}^t from (16) for f_F^t in the previous equation gives

$$\frac{1}{\eta}\left(FTP2 + \frac{M_s + M_G}{Z_b}\right) C_t + \frac{1}{\eta}\left(FCP2 - \frac{M_s + M_G}{Z_t}\right) C_b = f_{au}$$

or

$$f_{au} = \frac{1}{\eta}\left(FCP2 \; C_b + FTP2 \; C_t\right) \tag{17}$$

Similarly, substituting $f_{F\ell}^b$ from (11) for f_F^b and $f_{F\ell}^t$ from (15) for f_F^t gives an expression for the minimum average concrete stress produced by the minimum permissible prestress force at the lower limiting eccentricity.

$$f_{a\ell} = FCP1 \; C_t + FTP1 \; C_b \tag{18}$$

Substituting f_{au} and $f_{a\ell}$ for f_a in (14) will yield the upper and lower limiting eccentricities, respectively.

Position of the Neutral Axis

The position of the neutral axis or the value of y_b is governed by three factors: y_m, the minimum allowable value of y_b based on the requirements of the top section modulus; y_e, the minimum allowable value of y_b

based on the physical limitations of the cross section and cover requirements; and y_u, the minimum permissible value of y_b based upon ultimate strength requirements.

The minimum value of y_b, y_m, exists when both section moduli are at their minimums. Recalling that $C_b = \frac{y_b}{H}$ and $C_t = \frac{y_t}{H} = 1 - C_b$ and that $Z_bC_b = Z_tC_t$, an expression for the minimum allowable C_b and thus the minimum y_b or y_m will be derived.

$$C_bZ_b = (1 - C_b)Z_t$$

Solving for C_b in terms of Z_t and Z_b yields

$$C_b = \frac{Z_t}{Z_b + Z_t}$$

and setting Z_b and Z_t to their minimum allowable values as in Eqs. (5) and (6) gives

$$(C_b)_{min} = \frac{\eta FCP1 - FTP2}{\eta(FCP1 - FTP1) + FCP2 - FTP2}$$

so that

$$y_m = (C_b)_{min}\ H$$

In the case of $y_b < y_m$, the top section modulus will be less than the minimum required in equation (6) and it will be impossible to realize the required stress allowables for the top fiber.

In members with longer spans and curved tendons, y_b is governed mostly by the physical limitations of the section; i.e., the limiting eccentricities tend to fall outside the physical boundaries of the section, and y_b must be adjusted to remedy this situation. With an estimated y_s value, $y_e = e_s + y_s$ may be expressed by a quadratic equation of very complex coefficients, but Wang (3,4) has shown that it is better to use a trial method by tentatively selecting a y_b value and comparing it with $e_s + y_s$ to see if the tendon will fit into the section. If it will not, y_b is set to $e_s + y_s$ and the procedure repeated.

The third factor governing y_b is y_u based upon ultimate strength considerations. When the section has been designed by working stress theory and fails to meet ultimate strength requirements, the top flange width must be increased. This increase is accomplished by increasing the value of y_b to such a value y_u as to give the top flange the required width to meet ultimate strength requirements. As with the required value of y_e, it is also more practical to redesign the section if it fails to meet ultimate strength requirements based on a y_u slightly larger than the y_b of the previous design.

When an increase in the top flange width is not sufficient to meet the ultimate strength requirements, the area of prestressing steel may be increased until ultimate strength requirements have been met.

Required Flange Widths

When all dimensions except BT and BB are determined from the conditions of shear, deflection, fabrication, space available, etc., Guyon (6) has shown that the unknown flange widths, BT and BB, may be determined by solving two simultaneous equations involving the moment of inertia and moment-area requirements of the section. $I_{REQ'D} = Z_b \cdot y_b$, while the moment-area about the neutral axis is set to zero.

From Figure 5

moment of inertia

$$\frac{BT \cdot TT^3}{12} + BT \cdot TT \cdot CENTT^2 + \frac{(BT-BW)CT^3}{36} + \frac{(BT-BW)CT \cdot CENCT^2}{2}$$

$$+ \frac{BW \cdot HW^3}{12} + BW \cdot HW \cdot CENTW^2 + \frac{(BB-BW)CB^3}{36} + \frac{(BB-BW)CB \cdot CENCB^2}{2}$$

$$+ \frac{BB.TB^3}{12} + BB \cdot TB \cdot CENTB^2 = I_{REQ'D}$$

moment-area

$$BT \cdot TT \cdot CENTT + (BT-BW)\frac{CT}{2} . CENCT + BW.HW \cdot CENTW$$

$$-(BB-BW) \cdot \frac{CB}{2} \cdot CENCB - BB \cdot TB \cdot CENTB = 0$$

Now letting

$$D11 = \frac{TT^3}{12} + TT \cdot CENTT^2 + \frac{CT^3}{36} + \frac{CT}{2} \cdot CENCT^2$$

$$D12 = \frac{TB^3}{12} + TB \cdot CENTB^2 + \frac{CB^3}{36} + \frac{CB}{2} \cdot CENCB^2$$

$$D21 = TT \cdot CENTT + \frac{CT}{2} \cdot CENCT$$

$$D22 = TB \cdot CENTB + \frac{CB}{2} \cdot CENCB$$

$$C1 = I_{REQ'D} + BW\ (CT^3 + CB^3 + \frac{CB}{2} \cdot CENCB^2 + \frac{CT}{2} \cdot CENCT^2$$

$$- \frac{HW^3}{12} - HW \cdot CENTW^2)$$

$$C2 = BW\ (\frac{CT}{2}\ CENCT - HW \cdot CENTW - \frac{CB}{2}\ CENCB)$$

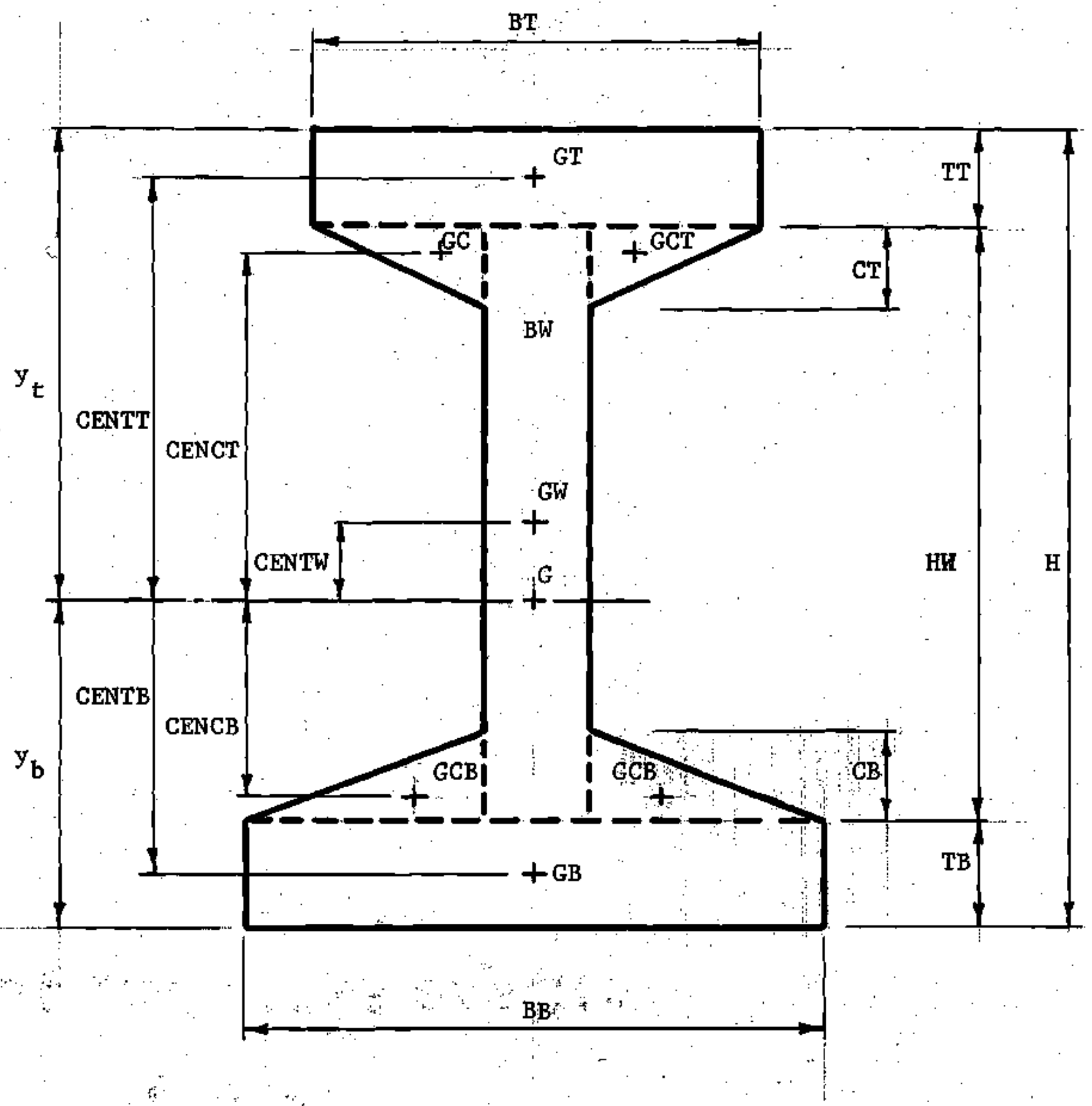


Figure 5. General Cross Section

The system of two equations and two unknowns may be expressed as

$$\begin{bmatrix} D11 & D12 \\ D21 & -D22 \end{bmatrix} \begin{matrix} BT \\ BB \end{matrix} = \begin{matrix} C1 \\ C2 \end{matrix}$$

Taking the determinant of the coefficient matrix

$$DET = -D11 \cdot D22 - D21 \cdot D12$$

Now solving for BT and BB

$$BT = (-C1 \cdot D22 - C2 \cdot D12)/DET$$

$$BB = (C2 \cdot D11 - C1 \cdot D12)/DET$$

Similar expressions for the unknown top flange width, BT, and the unknown web thickness, BW, of a T section may be determined by letting TB, CB, and BB go to zero in the moment of inertia and moment-area relationships derived for the I section and solving for BW and BT. Letting

$$D11 = \frac{TT^3}{12} + TT \cdot CENTT^2 + \frac{CT^3}{36} + \frac{CT}{2} \cdot CENCT^2$$

$$D12 = \frac{HW^3}{12} + HW \cdot CENTW^2 - \frac{CT^3}{36} - \frac{CT}{2} \cdot CENCT^2$$

$$D21 = TT \cdot CENTT + \frac{CT}{2} \cdot CENCT$$

$$D22 = HW \cdot CENTW - \frac{CT}{2} \cdot CENCT$$

$$C1 = I_{REQ'D}$$

$$C2 = 0$$

The system may be expressed as

$$\begin{bmatrix} D11 & D12 \\ D21 & D22 \end{bmatrix} \begin{matrix} BT \\ BW \end{matrix} = \begin{matrix} C1 \\ C2 \end{matrix}$$

The determinant of the coefficient matrix may be expressed as

$$DET = D11 \cdot D22 - D21 \cdot D12$$

Finally solving for BT and BW

$$BT = C1 \cdot D22/DET$$

$$BW = -C1 \cdot D21/DET$$

Design Procedure

Once certain section dimensions have been set by requirements of space available, shear resistance, form work, etc., stress requirements determined by a code or standard practice, and maximum midspan moments computed, the top and bottom flange widths may be directly determined. The following are the basic steps followed in the design of a precast concrete section and lend themselves to a programmed iterative design procedure:

1. Compute β and α based on the tendon profile requirements and loss factor.
2. Compute $(C_b)_{min}$ based on concrete stress allowables.
3. Initialize y_b to half the section depth.
4. Estimate M_G based on equations presented by Connolloy (7).

5. Compute the minimum required bottom section modulus based upon imposed moments and stress allowable considerations.
6. Determine the required moment of inertia from the product of y_b and $(Z_b)_{min}$.
7. Determine the required top and bottom flange widths by solving simultaneous moment-area and moment of inertia relationships.
8. Recompute M_G based on the new area of the section
9. Solve for f_{au} and $f_{a\ell}$ then substitute into the steel eccentricity relationship to find e_u and e_ℓ.
10. Set the steel eccentricity (e_s) to $y_b - y_s$, where y_s is the minimum required distance from the bottom fiber to the c.g.s.
11. If e_s is less than the required minimum (e_u), set $e_s = e_u$ and revise y_b to equal $e_s + y_s$.
12. If e_s is greater than the required maximum (e_ℓ), set $e_s = e_\ell$ and revise $y_b = e_s + y_s$.
13. Check y_b against its minimum allowable value, y_m. If y_b is less than y_m, set $y_b = y_m$.
14. If the value of y_b was changed in steps 11, 12, or 13, or if the value of M_G has been changed significantly from the last iteration, redesign the section starting with step 5.
15. Check the ultimate flexural capacity of the section; if the section fails to meet the ultimate capacity requirements, increase the value of $y_b = y_m$ and repeat the design procedure starting with step 5.

At this point, the section will meet working stress and ultimate strength requirements with the bottom section modulus at a minimum.

Design of Composite Sections

The method for designing a composite section with a previously designed slab is similar to the method previously presented for the design of non-composite members. The objective now is to select a prestressed concrete section with a suitable y_b to make the bottom section modulus of the composite section, Z_{bc}, a minimum value.

Equations similar to those for non-composite action will be developed for the general case in which: (1) the girder section is precast and fully prestressed (transfer); (2) it is then erected in place and the weight of the cast-in-place concrete added (casting operation); and (3) the balance of the superimposed load is added after the composite section has been formed (working load).

The full weight of the cast-in-place concrete will be carried by the bare girder, but the balance of the superimposed load will be resisted by the composite section. Sometimes the girder may be shored during erection so that the weight of the cast-in-place portion will also be resisted by the composite section. To account for this condition, the moment due to the weight of the cast-in-place slab, M_c, will equal zero and M_s will include the total load after erection.

Governing Equations

As shown for non-composite sections, the governing equations at transfer are

$$FCP1 \geqq F_o e_s/Z_b + F_o/A - \beta M_G/Z_b \qquad (19)$$

$$FTP1 \leqq -F_o e_s/Z_t + F_o/A + \beta M_G/Z_b \qquad (20)$$

and at working load after losses

$$FTP2 \leqq \eta F_o e_s/Z_b + \eta F_o/A - M_G/Z_b - M_c/Z_b - M_s/Z_{bc} \tag{21}$$

$$FCP2 \geqq -\eta F_o e_s/Z_t + \eta F_o/A + M_G/Z_t + M_c/Z_t + M_s/Z_{tc} \tag{22}$$

where Z_{bc} and Z_{tc} are the bottom and top section moduli, respectively, of the composite section.

Minimum Section Moduli

In order that all four inequalities may be based on the properties of the precast member, the terms k_b and k_t will be introduced:

$$k_b = Z_b/Z_{bc} \qquad \text{and} \qquad k_t = Z_t/Z_{tc}$$

For the bottom fiber, multiplying inequality (19) by η and (21) by -1 and adding

$$Z_b \geqq \frac{k_b M_s + M_c + \alpha M_G}{\eta FCP1 - FTP2} \tag{23}$$

and for the top fiber, multiplying inequality (20) by $-\eta$ and adding to inequality (22)

$$Z_t \geqq \frac{k_b M_s + M_c + \alpha M_G}{FCP2 - \eta FTP1} \tag{24}$$

Prestressing Force and Its Eccentricity

For $Z_b \geqq (Z_b)_{min}$ inequality (23) yields

$$\eta FCP1 - FTP2 \geqq \frac{k_b M_s + M_c + M_G}{Z_b} - \frac{\eta\beta M_G}{Z_b}$$

or

$$FCP1 + \frac{\beta M_G}{Z_b} \geqq \frac{1}{\eta}\left(\frac{k_b M_s + M_c + M_G}{Z_b} + FTP2\right) \tag{25}$$

and similarly for $Z_t \geqq (Z_t)_{min}$ inequality (24) yields

$$\frac{1}{\eta}\left(FCP1 - \frac{k_t M_s + M_c + M_G}{Z_t}\right) \geqq FTP2 - \frac{\beta M_G}{Z_t} \tag{26}$$

As may be seen from Figure 6

$$\frac{1}{\eta}\left(FCP2 + \frac{k_b M_s + M_c + M_G}{Z_b}\right) \leqq f_F^b \leqq FCP1 + \frac{\beta M_G}{Z_b}$$

and similarly

$$-FTP1 + \frac{M_G}{Z_t} \leqq f_F^t \leqq \frac{1}{\eta}\left(FCP2 - \frac{k_t M_s + M_c + M_G}{Z_t}\right)$$

The minimum permissible prestress force at the lower limiting eccentricity is limited by:

1) the final tension allowable in the bottom fiber

2) the initial tension allowable in the top fiber or, as seen from Figure 6,

$$f_{F1}^b = \frac{1}{\eta}\left(FTP2 + \frac{M_G + M_c}{Z_b} + \frac{k_b M_s}{Z_b}\right)$$

$$f_{F1}^t = FTP1 - \frac{\beta M_G}{Z_t}$$

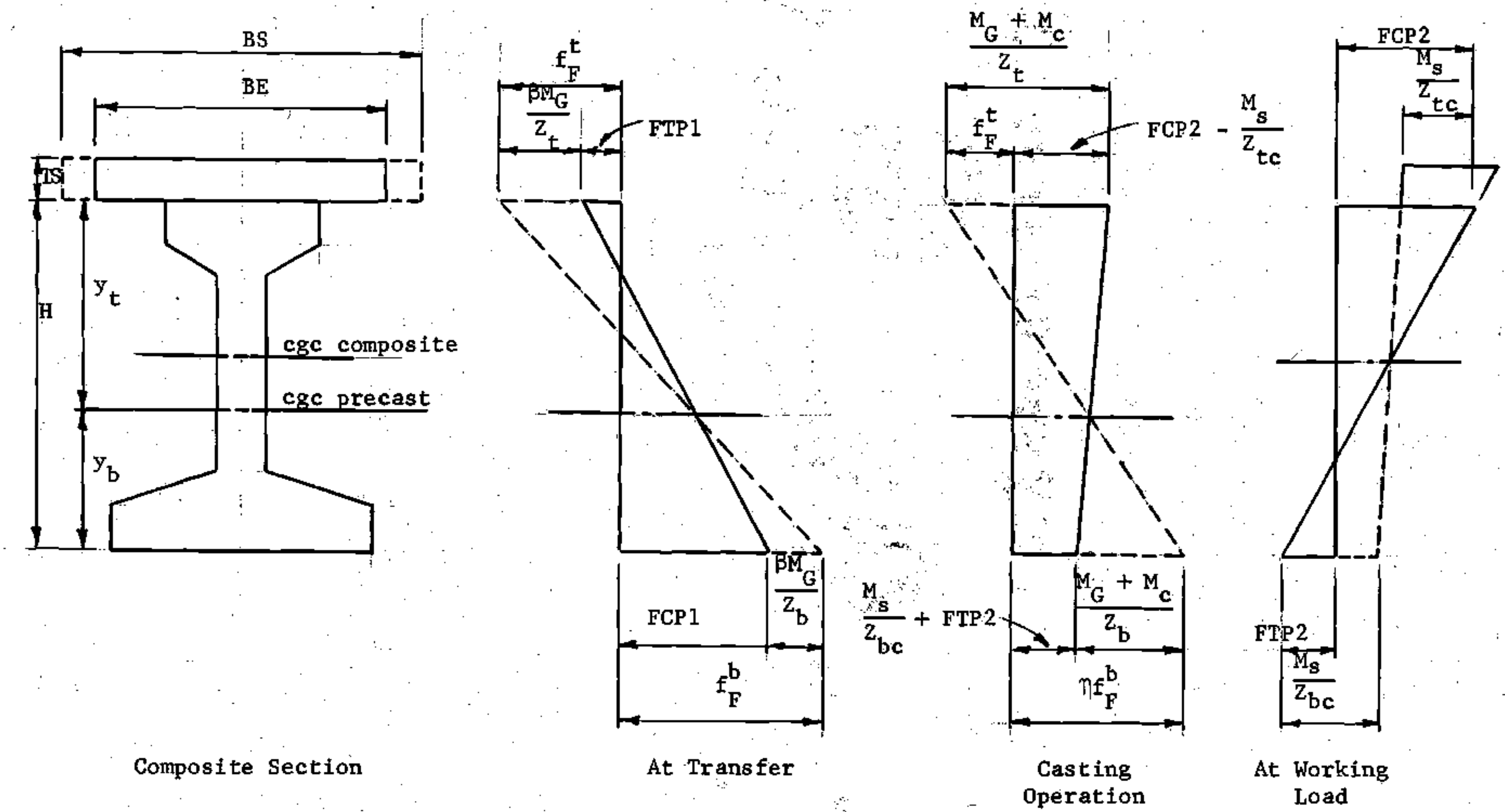


Figure 6. Composite Section Stress Distribution

The minimum permissible prestress force and corresponding eccentricity is limited by

1) the initial compression allowable in the bottom fiber
2) the final compression allowable in the top fiber

or,

$$f_{Fu}^{b} = FCP1 + \frac{\beta M_G}{Z_b}$$

$$f_{Fu}^{t} = \frac{1}{\eta}\left(-\frac{M_G + M_c}{Z_t} - \frac{k_t M_s}{Z_t} + FCP2\right)$$

Now recasting (25) and (26) as bounded inequalities

$$f_{F\ell}^{b} = \frac{1}{\eta}\left(FTP2 + \frac{k_b M_s + M_c + M_G}{Z_b}\right) \leqq f_F^b \leqq FCP1 + \frac{\beta M_G}{Z_b} = f_{Fu}^{b} \tag{27}$$

$$f_{F\ell}^{t} = FTP1 - \frac{\beta M_G}{Z_t} \leqq f_F^t \leqq \frac{1}{\eta}\left(FCP2 - \frac{k_t M_s + M_c + M_G}{Z_t}\right) = f_{Fu}^{t} \tag{28}$$

Following the same argument presented in the case of the non-composite section by setting Z_b to its minimum allowable as permitted by equation (5), an expression for the prestress eccentricity is formed which is the same as equation (13) for the non-composite section and an expression for f_a in terms of e_s is formed which is the same as equation (14). Again following the same argument as presented for the non-composite section, an expression may be derived for the average stress due to the prestressing force at its lower limiting eccentricity. This expression is the same as Eq. (18). Since f_{auc}, the maximum average stress when $e_s = e_u$ in the precast beam of a composite section, is affected by the loads imposed after composite action has taken place, the expression for f_{auc} is

$$f_{auc} = \frac{1}{\eta}\left[\left(FCP2 - \frac{k_b M_s}{Z_b}\right)C_b + \left(FTP2 - \frac{k_t M_s}{Z_t}\right)C_t\right] \tag{29}$$

The upper limiting eccentricity (e_u) is determined by substituting f_{auc} from (29) into Eq. (13) for f_a and the lower limiting eccentricity (e_1) similarly determined by substituting f_{a1} from (18) into Eq. (13) for f_a.

Required Flange Widths

The equations used to determine the required top and bottom flange widths are the same as those derived for non-composite sections. The required moment of inertia used in these equations is determined as the product of Z_{bc} and y_b.

Design Procedure

The design procedure used for composite section design is similar to that set forth for the non-composite section previously outlined. The only real difference between the two procedures is the introduction of the two terms, k_b and k_t, used in the equations derived for composite section design. Using the same basic design steps presented for the non-composite section, some additional steps will be added to account for k_b and k_t as follows:

4a) Initialize $k_b = 0.62$ and $k_t = 0.15$ where the initialized values represent typical average values for k_b and k_t.

14a) Compute $k_b = Z_b/Z_{bc}$ and $k_t = Z_t/Z_{tc}$. If either k_b or k_t differs significantly from its previously computed value, return to step 5 with their new values and redesign.

Ultimate Strength Capacity

The following procedure developed for the determination of ultimate strength capacity is developed to account for either composite or non-composite action and for a generalized I, T, or rectangular section.

AASHO specifications apply only to rectangular sections. These specifications are used where they apply, otherwise the internal stress couple approach is used to determine the ultimate moment with f_{su}, the average stress in the prestressing steel at ultimate load, calculated by the formula in section 1.6.10 of the AASHO code. A rectangular stress block is used throughout.

Computation of the ultimate capacity of a generalized section will be separated into four cases: (1) when the compressive block extends out of the slab; (2) when the stress block extends out of the slab but not into the tapered portion of the flange; (3) when the stress block is in the tapered section of the top flange; and (4)when the compressive stress block extends into the web.

Now referring to Figure 7

b = effective width of the area acted upon by the compressive stress block. May be "BE" for composite sections, "BT" for flanged sections, or "BB" for rectangular sections.

C1 = net compressive force developed if the stress were to cover the entire slab.

$= 0.85\ f'_c\ TS\ BE$

C2 = net compressive force developed if the stress block were to cover the non-tapered portion of the top flange only.

$= 0.85\ f'_c\ (TT - GS)\ BT$

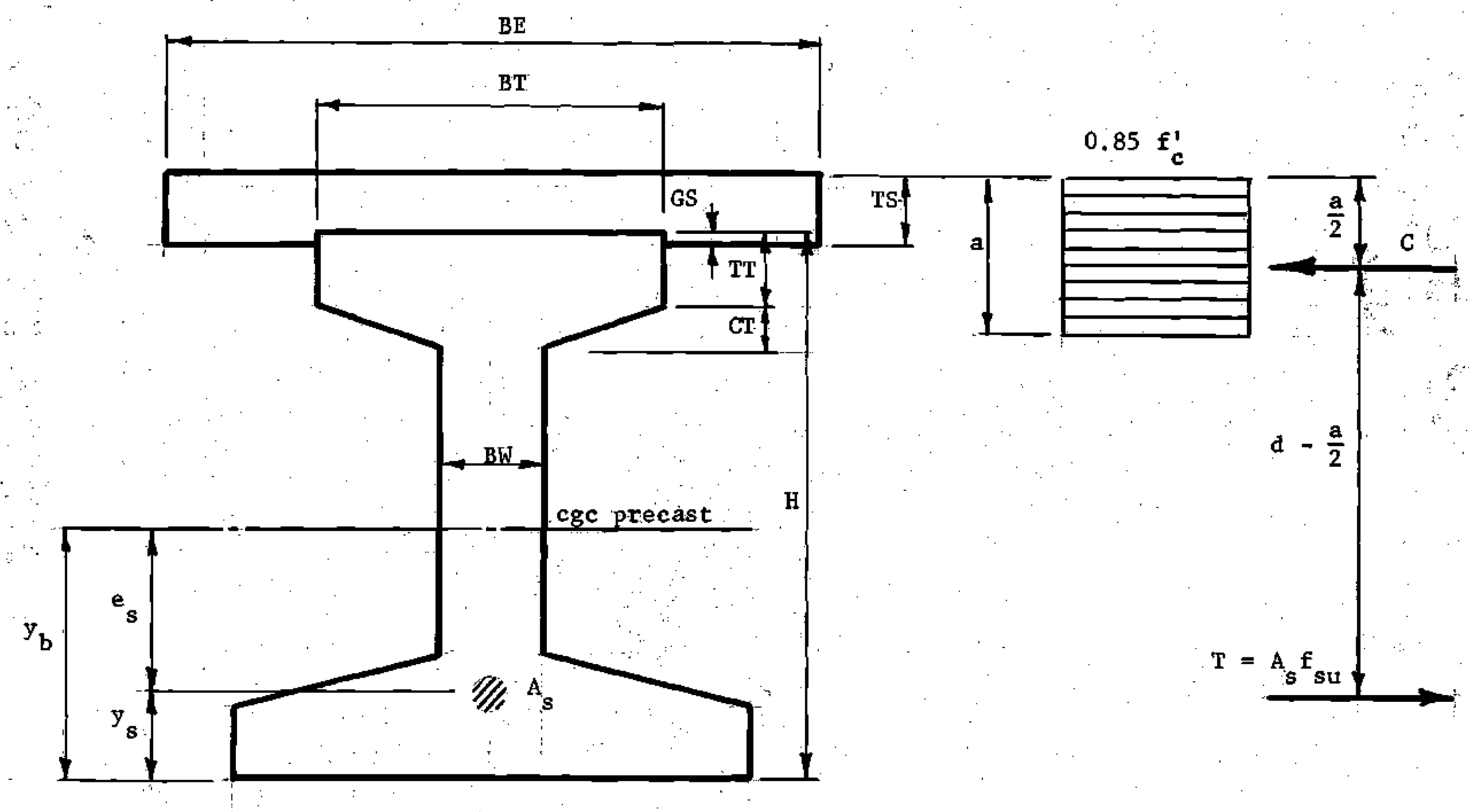


Figure 7. Generalized Section and Stress Block

C3 = net compressive force developed if the stress block were to cover the tapered portion of the flange only.

$= 0.85\ f'_c\ (BT + BW)\ CT/2$

d = the distance from the extreme compressive fiber to the centroid of the prestressing force.

$= H - y_s + (TS - GS)$

Let C_1^*, C_2^*, and C_3^* be the compressive force developed if the compressive force were to cover areas 1, 2, and 3, respectively of Figure 9.

Using the AASHO code requirements for the stress in the prestressing steel at ultimate load for bonded tendons, we have

$$f_{su} = f'_s\ (1.0 - 0.5\ p\ f'_s/f'_c)$$

where f'_s is the ultimate strength of the prestressing steel, $p = A_s/bd$, the ratio of prestressing steel, and f'_c is the 28 day compressive strength of the concrete. Similarly for unbonded tendons

$$f_{su} = f_{se} + 15{,}000$$

where f_{se}, the effective steel prestress after losses, is not greater than $0.6\ f'_s$ or $0.8\ f_{sy}$, and f_{sy} is the nominal yield point stress of prestressing steel (at 1.0 percent extension).

By comparing the ultimate tendon capacity $T = A_s\ f_{su}$ with C1, C1 + C2, or C1 + C2 + C3, a determination is made as to the depth of the rectangular stress block for a given section. For example, if $C1 < T < C1 + C2$, the stress block extends into the non-tapered portion of the flange, i.e., case 2, and the ultimate flexural capacity may be computed on that basis.

Case 1

For the first case the compression block remains within the slab as determined by $C1 \leqq T$. In this case the ultimate flexural capacity of the section, M_u', may be expressed as

$$M_u' = A_s f_{su} (d - 0.59qd)$$

where $q = p f_{su}/f_c'$, except that the AASHO code provides that if $q > 0.3$, the ultimate moment capacity shall be expressed as

$$M_u' = 0.25 f_c' bd^2$$

Case 2

For this case, the stress block extends into the top flange and A_{ss} is defined as the area of prestressing steel required to balance C_1^* and A_{sr} as the steel remaining determined from $A_{sr} = A_s - A_{ss}$. Using an internal couple approach, the ultimate capacity of the section is seen to be

$$M_u' = A_{ss} f_{su}(d - TS/2) + A_{sr} f_{su}(d - 0.59qd)$$

or

$$M_u' = A_{ss} f_{su}(d - TS/2) + 0.25 f_c' bd^2$$

in the case of $q > 0.3$.

Case 3

Case 3 occurs when $C1 + C2 \leqq T \leqq C1 + C2 + C3$ or when the compression block extends into the tapered portion of the top flange. The internal couple approach used in cases 1 and 2 may be found in most texts treating ultimate strength theory, but case 3 presents some special problems worth considering.

The depth to which the stress block extends into the taper is not at first apparent. Let $\Delta = T - C1 - C2$, then Δ is the net compressive force resisted by the tapered section of the beam only. From Figure 8, by taking similar triangles

$$x = k(BT - BW)/2CT$$

After determining the shaded area as a function of k,

$$A(k) = k(BT - 2x) + kx$$

and substituting for x

$$A(k) = k\,BT - k^2(BT - BW)/2CT$$

we may not write Δ as

$$\Delta = 0.85\ f'_c\ A(k)$$

and solving for k yields

$$k = \frac{BT + \sqrt{BT^2 - 2.35(BT-BW)\Delta/CT}}{(BT-BW)/CT}$$

Letting A_{ss} be the area of prestressing steel required to balance the resultant compressive force over area 1 of Figure 9, A_{sf} the area of steel required to balance the resultant over area 2, and A_{sr} the area remaining, yields

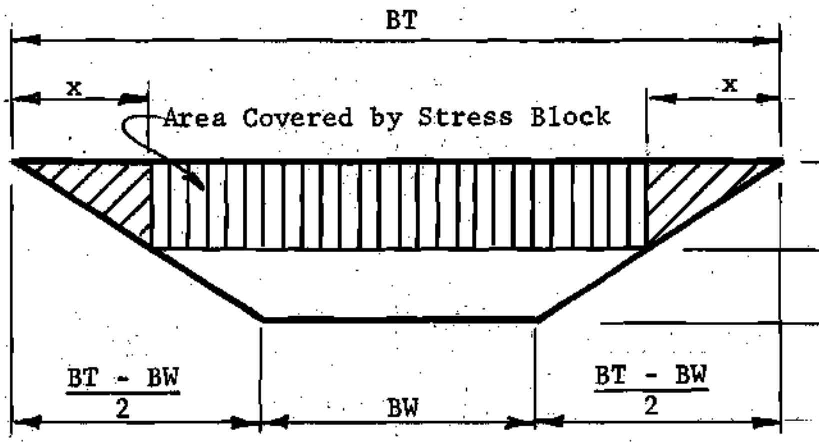


Figure 8. Tapered Portion of Beam

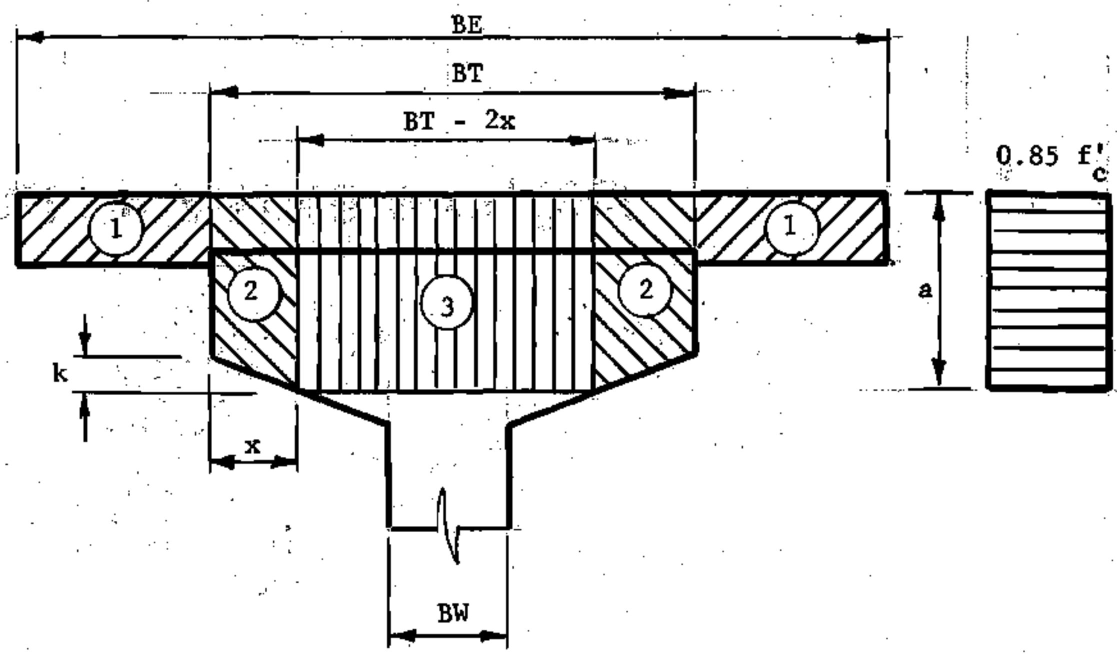


Figure 9. Compressive Stress Areas

$$A_{ss} = 0.85\ f'_c\ (BE - BT)TS/f_{su}$$

$$A_{sf} = 0.85\ f'_c(2x)(TS + TT - GS + 0.5k)/f_{su}$$

$$A_{sr} = A_s - A_{ss} - A_{sf}$$

Summing horizontal forces over area 3 to zero gives

$$A_{sr}\ f_{su} = 0.85\ f'_c\ a(BT - 2x)$$

or

$$a = \frac{A_{sr}\ f_{su}}{0.85\ f'_c\ (BT - 2x)}$$

but since $p = A_{sr}/(BT - 2x)d$ and $q = p\ f_{su}/f'_c$, one may write $a/2 = 0.59qd$.

Consider the ultimate couples of Figure 10 where C^*_1, C^*_2, and C^*_3 are the compressive forces acting over compressive blocks 1, 2, and 3, respectively, as shown in Figure 9. All quantities are known except the centroidal distance of compression block 2 from the top fiber, CENT. To determine CENT the total area of block 2 is expressed as

$$A_2 = x(TS + TT - GS) + kx/2$$

and the moment-area of block 2 about the top fiber

$$MA_2 = 0.5x(TS + TT - GS)^2 + 0.5kx(TS + TT - GS + k/3)$$

so that

$$CENT = MA_2/A_2$$

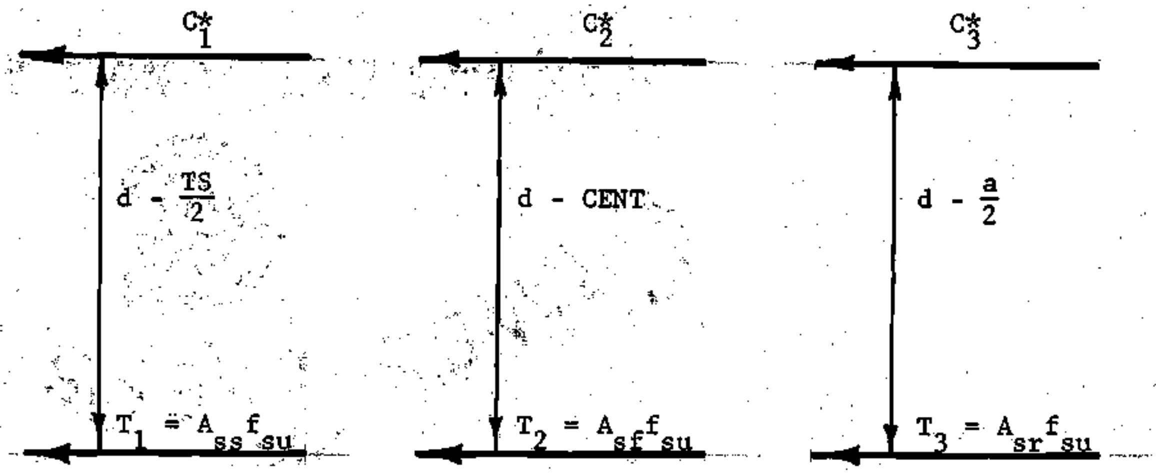


Figure 10. Ultimate Couples

Now summing the ultimate couples shown in Figure 10

$$M_u' = A_{ss}\ f_{su}(d - TS/2) + A_{sf}\ f_{su}(d - CENT) + A_{sr}\ f_{su}(d - a/2)$$

unless $q > 0.3$, then

$$M_u' = A_{ss}\ f_{su}(d - TS/2) + A_{sf}\ f_{su}(d - CENT) + 0.25\ f_c'\ bd^2$$

Case 4

When T is greater than C1 + C2 + C3, the compression block extends into the web of the section. For this case

$$M_u' = A_{ss}\ f_{su}(d - TS/2) + A_{sf}\ f_{su}(d - CENT) + A_{sr}\ f_{su}(d - a/2)$$

where CENT is determined as in case 3 with $k = CT$ and $x = BT - BW$. As previously when $q \geqq 0.3$

$$M_u' = A_{ss}\, f_{su}(d - TS/2) + A_{sf}\, f_{su}(d - CENT) + 0.25\, f_c'\, bd^2$$

CHAPTER IV

COMPUTER PROGRAM CAPABILITIES AND METHODS OF COMPUTATION

The basic purpose of the computer program described herein is the design of simple spanned prestressed concrete highway bridge girders utilizing the methods developed in the previous chapters. While certain features of the program (AASHO truck loadings, AASHO specifications, etc.) are directed to this end, the program may also be applied to the design and analysis of any simple spanned prestressed concrete beam. Not only is the program applicable to different types of girders, but it may also be used to check a user input girder for violation of stress, tendon profile, ultimate strength, or deflection requirements. In both the design mode and check mode of the program, the user has the option of specifying a number of separate loadings for which a maximum moment envelope is generated. This moment envelope is used to determine either a beam design or the adequacy of a given beam section. The user is required to specify the number of segments (NSEG) into which the span length is equally divided. Including the sections at both ends of the spans, there are NSEG + 1 equally spaced sections cut along the length of the span. It is at each of these sections that moments, shears, net and transformed section properties, tendon eccentricities, stresses, and deflections are computed and output.

Loadings

The program loading specifications are tailored to standard AASHO loading types, but static loadings may be applied to any beams, ignoring the moving load capabilities, whether it is a highway bridge girder or not. The program also provides the means of including several loading combinations (up to five) of which the worst case of each of the loadings controls.

Static Loads

Three types of static loadings have been incorporated into the program: uniform loads, uniform segment loads, and concentrated loads. Each loading type may be included any number of times in a single loading combination. For example, loading combination number one may include a uniform load plus three concentrated loads. The moments and shears produced by static loads are generated at NSEG + 1 points along the span length. Moments and shears are computed at each section by considering each loading type within a loading combination separately then finally combining all such moments and shears after each has been computed individually.

Moving Loads

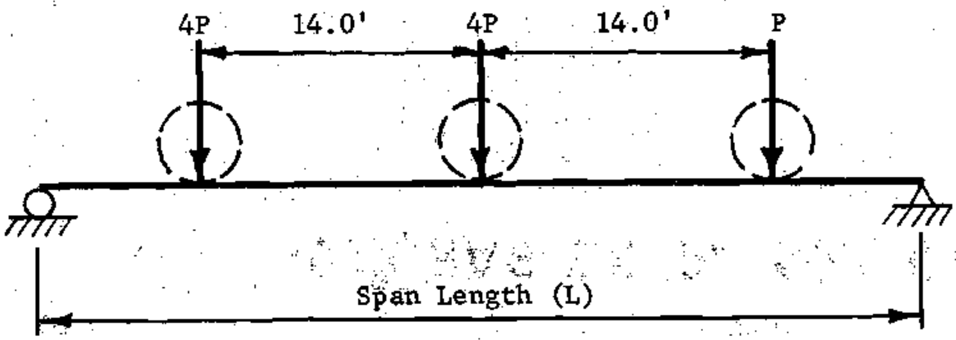
The computer program is specifically written to generate a moment envelope caused by a standard AASHO truck moving across the span or by an AASHO equivalent lane loading. The moment envelopes generated are modified by the program to include the effect of the number of beams supporting a single traffic lane. This modification is made by multiplying the moment envelope for a single lane by a user specified live load distribution coefficient. For example, if two beams support one traffic lane, the user specified distribution coefficient would be 0.5 and the moment envelope for a single girder would be half of that for a single traffic lane.

The maximum moment envelope generated by a truck moving across a simple span is determined by considering the moment generated under the center wheel of the standard AASHO truck at NSEG + 1 points along the span as the truck moves across the length of the span. The moments produced by this sequence do not necessarily generate the maximum moment envelope but rather produce an envelope of moments caused by a truck moving in one direction across the span. The true maximum moment envelope is generated by a truck moving in either direction across the span. To produce this maximum envelope, moments generated are checked for symmetry about the center of the span. Where symmetry is violated, the maximum ordinate is taken as the control and a symmetrical envelope generated. These are the moving load moments for design and analysis.

For example, consider the AASHO standard HS truck loadings represented in Figure 11. In order to generate the maximum moment envelope, three cases must be considered: case A, spans less than 14 feet; case B, spans greater than 14 feet but less than 28 feet; and case C, spans greater than 28 feet.

It should be noted that the variable rear axle spacing is placed at the minimum permissible value (14 feet) for all computations. This produces the maximum moment for simple span beams by concentrating as much load as possible near the center of the span.

<u>Case A</u> -- Let V(x) and M(x) represent the shear and moment as a function of the distance along the length of the span where x represents



P = 8000 lbs HS20-44

P = 6000 lbs HS15-44

Figure 11. Standard HS Truck Loadings

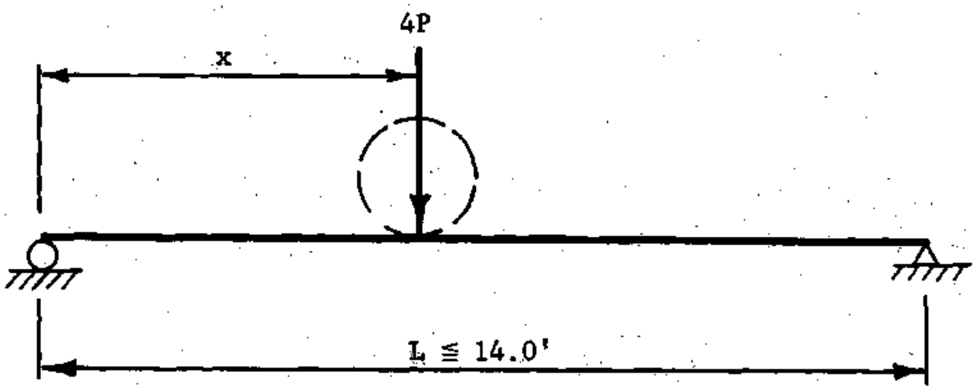


Figure 12. Case A — Spans Less Than 14 Feet

the distance from the beginning (left support) of the span to the center wheel load (see Figure 12) then,

$$V(x) = \frac{4P(L - x)}{L}$$

$$M(x) = V(x)\ x$$

<u>Case B</u> -- For spans greater than 14 feet but less than 28 feet (see Figure 13),

$$V(x) = \frac{8P(L - x - 7)}{L}$$

$$M(x) = V(x)\ x$$

until x = L - 14 feet (or for negative x), then case A controls.

<u>Case C</u> -- For spans greater than 28 feet (see Figure 14), the maximum moment envelope is generated under the middle wheel load at x while the maximum shear is developed under the end wheel load and may be expressed as

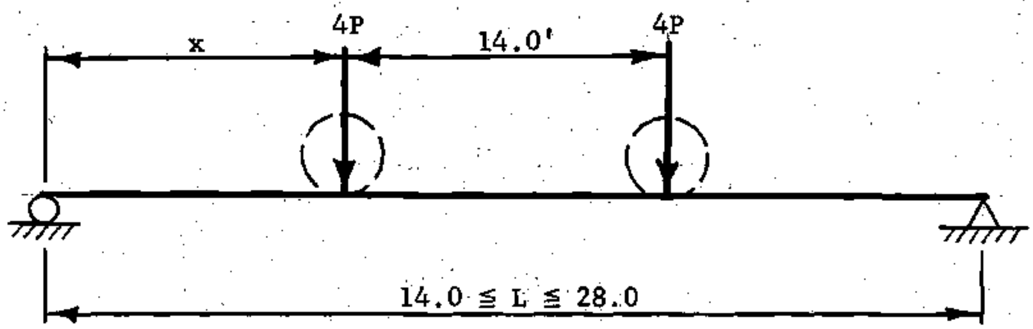
$$V(x) = \frac{3P}{L}(3L - 28 - 3x_1)$$

until x_1 = L - 28 feet, then case B controls. The moment may be expressed as

$$M(x) = \frac{3P}{L}(3L - 14 - 3x)x - 56P$$

until x = L - 14 feet, then case B controls.

Expressions may be developed for standard H truck loadings analogous to the method presented for the HS standard truck loadings and their



P = 8000 lbs HS20-44 P = 6000 lbs HS15-44

Figure 13. Case B — Spans Greater Than 14 Feet But Less Than 28 Feet

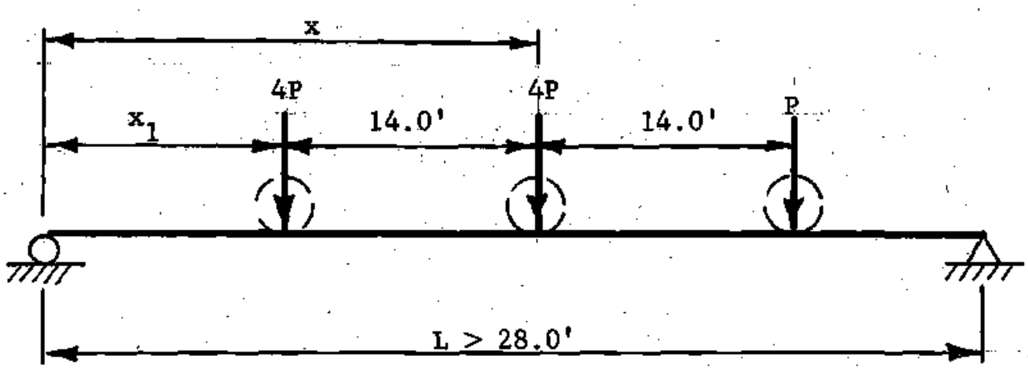


Figure 14. Case C — Spans Greater Than 28 Feet

derivations will not be presented here. Expressions for the H and HS equivalent lane loadings also follow similarly with a uniform load combined with a single concentrated load moving across the span generating a maximum moment envelope.

Moments produced by static loadings and moving loads for a single loading combination are combined to produce the total superimposed moment envelope that is used for the design or analysis of a prestressed concrete section.

Shears are computed and output by the program only for the convenience and information of the user. No attempt has been made in this program to either design or check for shear requirements.

Design Mode

The computations involved and steps taken in the design of a prestressed girder are presented in detail in Chapter III. It is this method that has been programmed to effect a design. After a girder has been designed, it is checked for violation of any of the user specified dimension restraints. Should any restraint be violated, the section is redesigned with appropriate adjustments to section dimensions intended to produce a design within the band of user specified constraints.

User Options

Under the design mode there are certain parameters which the user must specify and other parameters which are optional. The required parameters which the user must specify are: span length, composite or noncomposite section, I, T, or rectangular section, bonded or unbonded tendon, number of points into which the span is divided for moments and shears, tendon eccentricities, deflection computations, etc., and the slab dimensions in the case of composite action. These are the most essential parameters needed for design. If any are not specified, it will cause termination of the program. All other parameters, if left unspecified, will be assigned default values (see Appendix D) by the program or left unspecified (or zero).

Optional parameters include: all dimensions and dimension restraints, allowable concrete and steel stresses, tendon profile specifications, loss factor, density of concrete, minimum distance from the bottom fiber to the center of the prestressing steel, modulus of elasticity of the precast concrete and the poured-in-place slab, creep factor for computation of long term deflections, and whether the slab of a composite section is shored during construction.

Default Values

All optional parameters not specified are assigned default values by the program. In many cases the default values are the values that the user would normally specify. It is for this reason that the user need not specify many input parameters. For example, the stress allowables are assigned as per section 1.6.7 of the 1965 edition of the AASHO code specifications for highway bridges. The only reason then for the user to override the default values and input his own stress allowables is if he was not designing according to the AASHO code specifications or if he had sufficient reason to modify the required code values. In some cases the user may not want to specify any of the optional parameters, letting all optional parameters be assigned values by the program. For a complete listing of all default values, see Appendix D.

Output

The program has been written to output all pertinent design information. All input information, moments and shears, computed section dimensions, section properties, midspan stresses, tendon profile requirements, prestress force, area of steel, and deflections are typically output. This output is intended to be a diagnostic aid for the user in determining the parameter specification(s) which caused termination of the program.

Check Mode

When the user is not designing a girder to meet certain requirements but rather needs to check a girder for a set of conditions, he specifies the calling of the check mode of the program. Under the check mode, as under the design mode, a girder may be checked for adequacy under several different loading conditions. Input specifications are much the same as for the design mode and the output, although somewhat different in format, contains the same basic information. The only basic difference between the design mode and check mode is in the computation of section properties. Under the design mode, section properties are based on the gross section area and considered constant for the length of the span, while under the check mode, section properties are based on the net/transformed section areas which vary along the length of the span. A girder, for instance, may be designed under the design mode and this same design input under the check mode may be found to be slightly inadequate because of the difference in section properties. The last section in Chapter II discusses this problem.

Input Options

The primary difference between the input parameters specified under the design mode and the parameters specified under the check mode is in the number of required specifications. All the parameters required under the design mode are also required under the check mode but there are also additional parameters which were optional under the design mode that now become required under the check mode. All section dimensions which were optional under the design mode are now required input parameters and all section dimension restraints, optional under the design mode, become meaningless. The user must specify an initial prestress force and tendon eccentricity. If the section is to be checked using the net/transformed section properties and the ultimate flexural strength of the section is to be computed, the area of the steel is not specified. If the area of steel is not specified or purposely set to zero, the program is "tricked" into checking the section based on its gross section properties and the ultimate flexural strength is output as zero. All other parameters, if left unspecified, are assigned the same default values as assigned in the design mode (see Appendix D).

Output

The same information output under the design mode is also output by the check mode. This information sometimes, however, must assume a different type of output format. Stresses, for example, are output at NSEG + 1 points along the girder span length instead of just at midspan.

Section Properties

The method used to determine the section properties of a generalized section is based on the computation of the areas, centroidal distances, and moments of inertia of isolated sections. Consider the generalized I section shown in Figure 15. Let A_i be the area of section i, $CENT_i$ the centroidal distance from the bottom of the full section to the centroid of section i, and MOI_i the moment of inertia of section i about its neutral axis. It is now possible to write relationships for the areas and centroidal distances of the individual sections.

$$A_1 = BT\ TT \qquad CENT_1 = H - TT/2$$

$$A_2 = CT(BT - BW) \qquad CENT_2 = H - TT - CT/3$$

$$A_3 = BW(H - TT - TB) \qquad CENT_3 = (H - TT +)/2$$

$$A_4 = CB(BB - BW) \qquad CENT_4 = TB + CB/3$$

$$A_5 = TB\ BB \qquad CENT_5 = TB/2$$

The total area may be expressed as

$$AREA = \sum_{i=1}^{5} A_i$$

while the distance from the bottom fiber to the neutral axis, y_b, is

$$y_b = \sum_{i=1}^{5} A_i\ CENT_i/AREA$$

and $y_t = H - y_b$. The moment of inertia of each section is expressed as

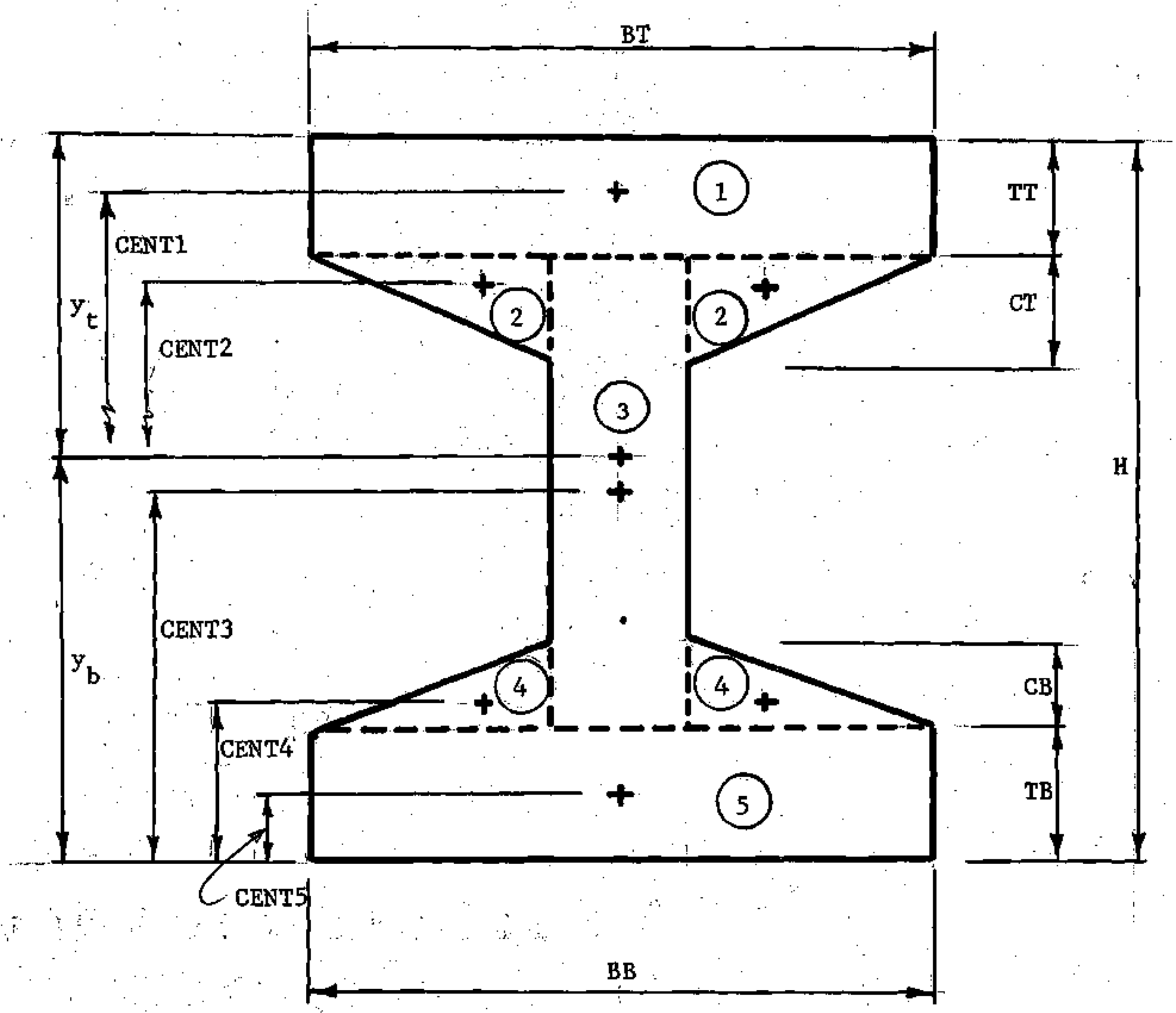


Figure 15. Geometry of General I Section

$$MOI_1 = BT\ TT^3/12$$

$$MOI_2 = (BT - BW)CT^3/36$$

$$MOI_3 = BW(H - TT - TB)^3/12$$

$$MOI_4 = (BB - BW)CB^3/36$$

$$MOI_5 = BB\ TB^3/12$$

and the total moment of inertia, I, is

$$I = \sum_{i=1}^{5} (MOI_i + A_i(CENT_i - y_b)^2)$$

The top and bottom section moduli may now be written as

$$Z_b = I/y_b \qquad \text{and} \qquad Z_t = I/y_t$$

Composite section properties are computed similarly to the properties previously shown for the non-composite section. Consider the composite section in Figure 16. Let AREAS = BE TS - GS BT be the effective area of the slab and CENTS the distance from the bottom fiber of the section to the section to the centroid of the composite section may be expressed as

$$AREAC = AREA + AREAS$$

and the distance from the bottom fiber to the neutral axis of the composite section, y_{bc}, is

$$y_{bc} = (AREA \cdot y_b + AREAS \cdot CENTS)/AREAC$$

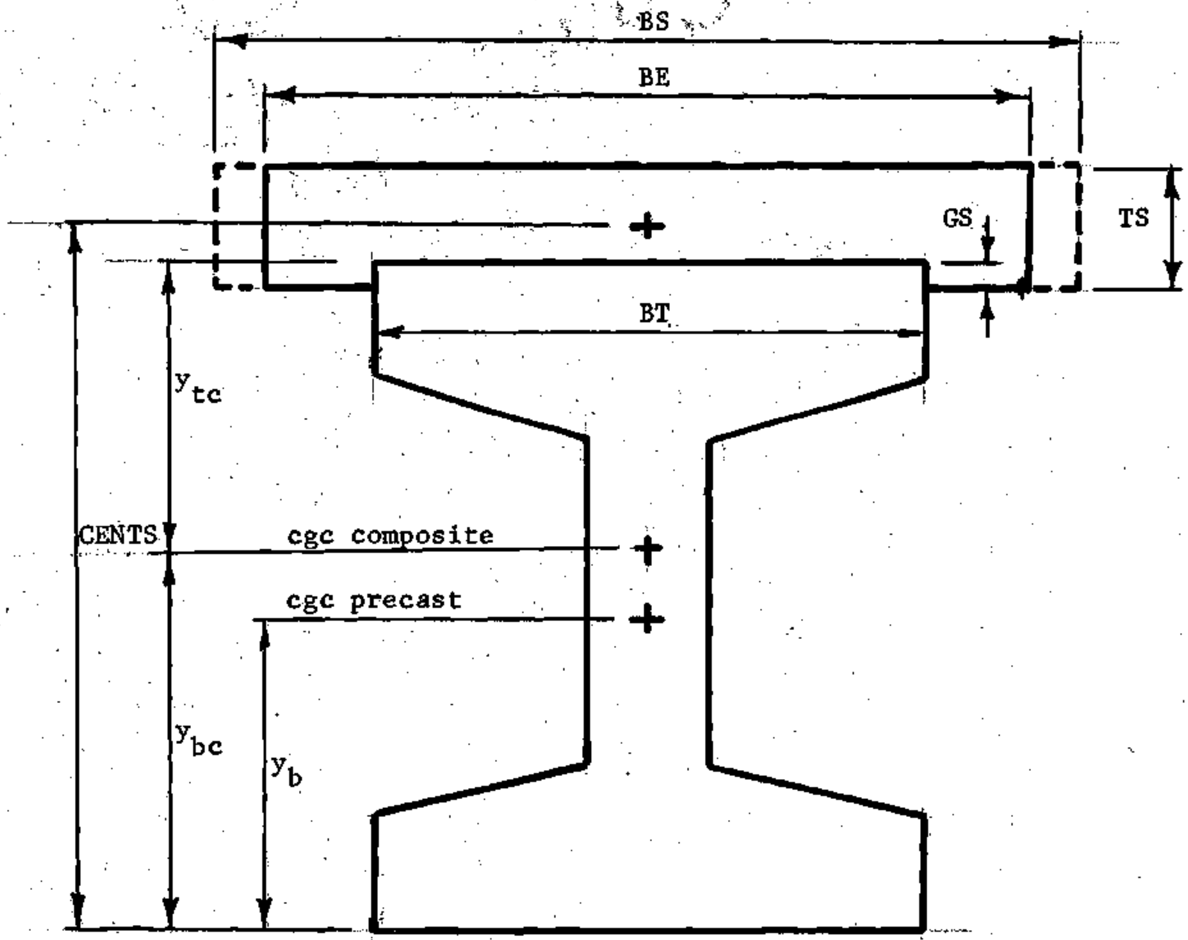


Figure 16. Geometry of Composite Section

and $y_{tc} = H - y_{bc}$. The moment of inertia of the slab about is centroid is

$$MOIS = (BE\ TS^3 - BT\ GS^3)/12 + BE\ TS(TS/2 - y_{bs})^2 - BT\ GS(y_{bs} - GS/2)^2$$

where y_{bs}, the distance from the bottom fiber to the neutral axis of the slab, is expressed as

$$y_{bs} = ((BE\ TS^2)/2 - (BT\ GS^2)/2)/AREAC$$

and the moment of inertia of the composite section may be expressed as

$$I_c = I + AREAT(y_{bc} - y_b{}^2) + MOIS + AREAS(H - GS - y_{bs} - y_{bc})^2$$

The section moduli may now be computed as

$$Z_{bc} = I_c/y_{bc} \qquad \text{and } Z_{tc} = I_c/y_{tc}$$

Deflections

Beam deflections are computed and output at NSEG + 1 points along the length of the span for each loading combination specified by the user. These deflections are based on the prestress force after losses and take into consideration creep (input by a user specified creep factor) associated with long time loads. One exception occurs, however, when the user specified zero loading under one of the loading conditions. In this case the initial prestress force is assumed and the initial camber due to prestressing is output. In the case of a moving truck load, the truck is positioned to produce a maximum moment at midspan.

Deflection Computations

Beam deflections are computed utilizing moment-area relationships based either on the gross section properties as in the case of computations carried out under the design mode or on the net/transformed section properties when a section is being examined by the check mode.

Consider some arbitrary loading and resulting moment diagram with a deflected shape as shown in Figure 17. By moment-area

$$y'(B) = y'(A) + \text{AREA } \frac{M}{EI}\Big|_A^B$$

and

$$t_{BA} = \text{AREA } \frac{M}{EI}\Big|_A^B X_B^*$$

where $y'(x)$ is the slope slope of the deflected shape at point x, t_{BA} is the tangential offset at a point B from the tangent at A, and X_B^* is the distance from the centroid of the $\frac{M}{EI}$ diagram to point B.

The moment diagram is divided into NSEG equal segments and the length of each segment is

$$XSEG = LEN/NSEG$$

The moment is assumed to vary linearly within a given segment, thus the area and centroid of each segment may be easily computed as seen from Figure 18. The area of each segment is expressed as

$$A(I) = 0.5\ XSEG(M(I) + M(I + 1))$$

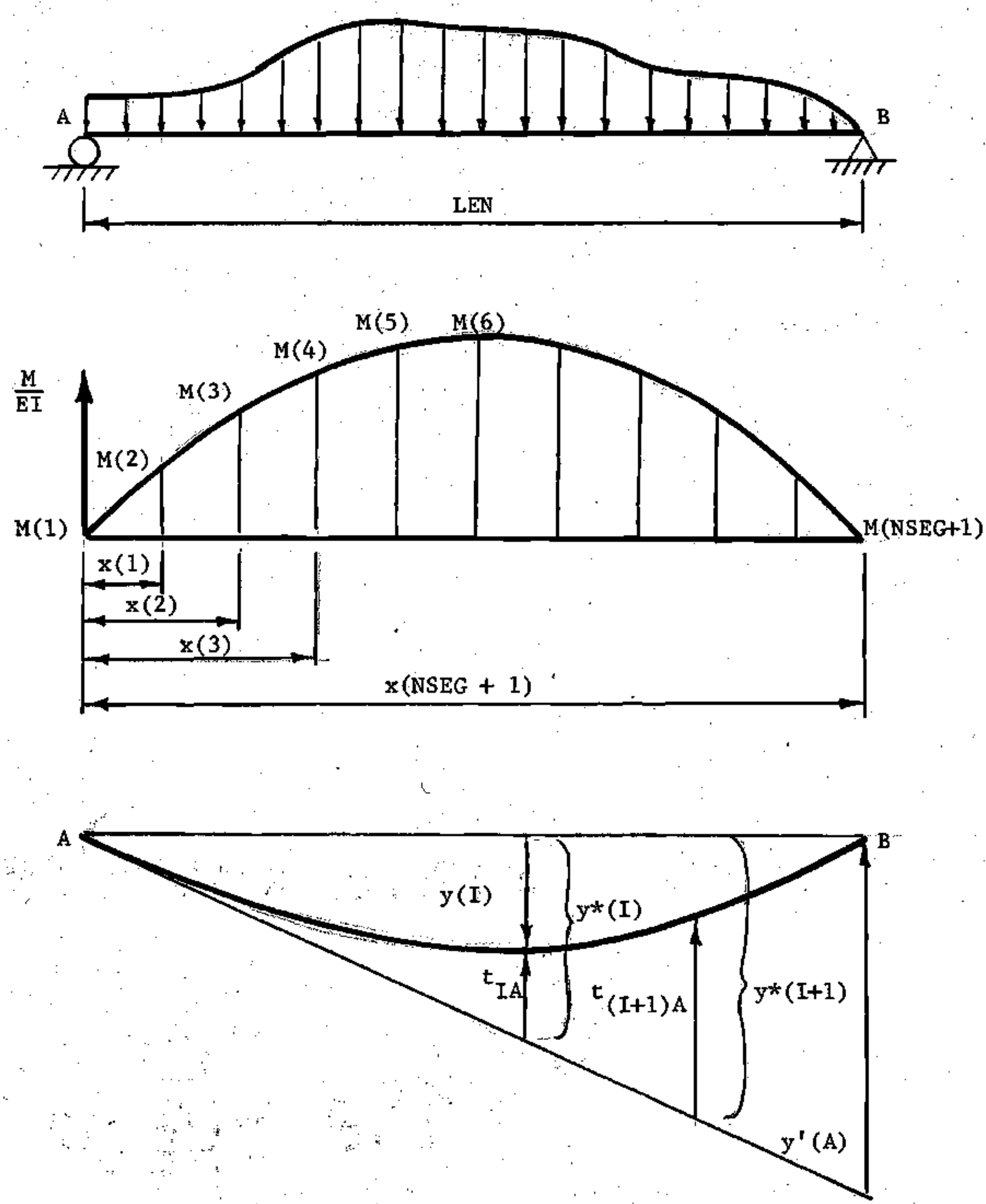


Figure 17. Loading, Moment, and Deflection Diagrams

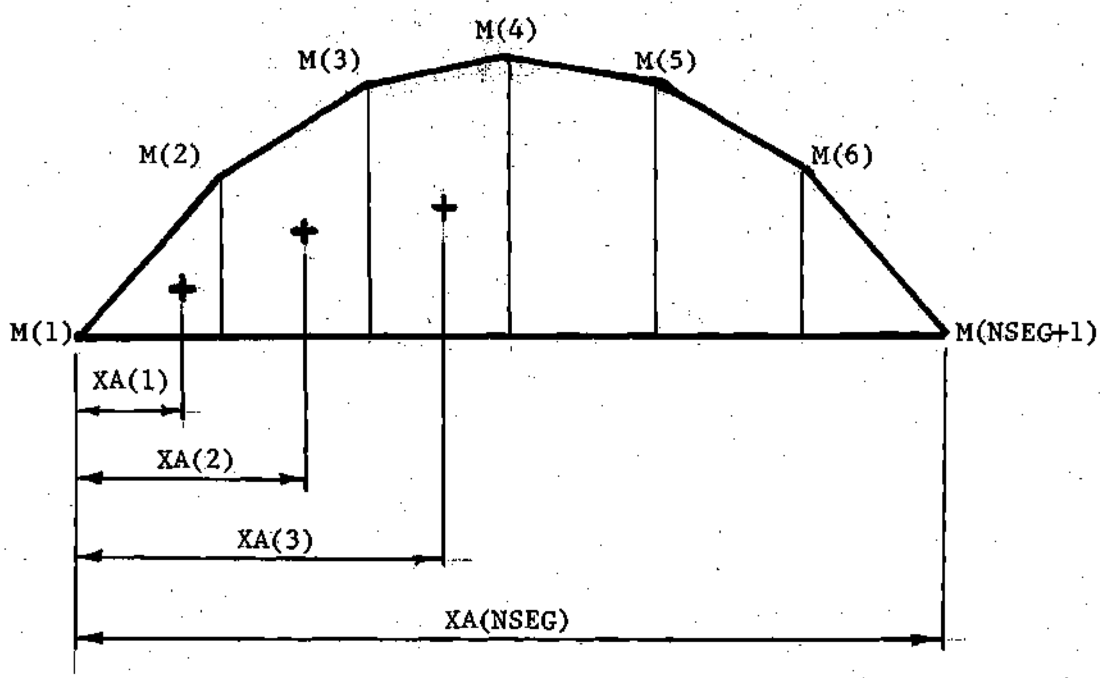


Figure 18. Linearized Moment Diagram

and the centroidal distance to area I is

$$XA(I) = XSEG(I - 1) + \frac{XSEG^2}{A(I)}\left(\frac{M(I + 1)}{3} + \frac{M(I)}{6}\right)$$

The tangent offset, t_{BA}, may now be computed as

$$t_{BA} = \sum_{I=1}^{NSEG} A(I)\ (LEN - XA(I))$$

and the slope of the deformed elastic curve at A is

$$y'(A) = -t_{BA}/LEN$$

while $y^*(I)$ is expressed as

$$y^*(I) = y'(A)(I - 1)XSEG$$

Knowing the slope at point A, one may compute the offset at point I from the tangent at A, t_{IA} (see Figure 17)

$$t_{IA} = \sum_{n=1}^{I-1} A(n)\ [X(I) - XA(n)]$$

and finally the deflection at I is expressed as

$$y(I) = y^*(I) - t_{IA}$$

Stresses

Stresses in the top and bottom fibers (see Eqs. 1, 2, 3, and 4) are computed in the design mode for initial and final conditions at mid-span based on the gross section properties while computations in the check mode are made at NSEG + 1 sections along the span and are based on the net/transformed section properties. The net section, which deducts the area cored out by the tendon, is used for stress computations due to the prestressing force and the dead weight of the girder. The transformed section, which transforms the area of prestressing steel into an equivalent concrete area, is used for stress computations involving loads superimposed after the initial prestressing.

CHAPTER V

PROGRAM USE AND EXAMPLE PROBLEM

The intent of this chapter is to acquaint the reader with the options of program use, a description of each of the input parameters and its use, a discussion of loading input, and a general description of a typical program input.

Input Parameters

The user is presented an array of parameters which he may specify as program input but seldom does he make use of all parameters for any single problem. Rather, he specifies only those parameters which apply to his particular problem and program use, letting many parameters assume default values (see Appendix D).

The following is a list of all input parameters.

Problem Description

DES	= 1,0 -- Call to design mode = 1, call to check mode = 0 (or unspecified)
COMP	= 1,0 -- Composite section = 1, non-composite section = 0 (or unspecified)
TYPE	= 1,2,3 -- I section = 1, T section = 2, rectangular section = 3
BOND	= 1,0 -- Bonded tendon = 1, unbonded tendon = 0 (or unspecified)

SUP = 1,0 -- Slab supported by scaffolding during erection = 1, slab supported only by girder during erection = 0 (or unspecified)

TENS = 1,2 -- pretensioned tendon = 1, post-tensioned tendon = 2

LEN = span length, any positive number (ft)

NSEG = number of segments into which the span is divided. Must be an even, positive integer under 50. Used to determine the points along the span for computation of moments, shears tendon eccentricity, profile limits and stresses.

Strengths and Stress Allowables

FCP = f'_c or 28 day concrete compressive strength (psi)

FCPI = f'_{ci} or initial concrete compressive strength (psi)

FSP = f'_s or ultimate strength of prestressing steel (psi)

FCP1(FTP1) = initial allowable compressive (tensile) stress in concrete (psi)

FCP2(FTP2) = final allowable compressive (tensile) stress in concrete (psi)

Tendon Description

TL = horizontal segment of a harped tendon, for a parabolic tendon = 0.0 (or unspecified) for a harped tendon = a value (in feet) between 0.0 and LEN, for a straight tendon = LEN (see Figure 3)

NEDA = fraction of initial prestress remaining at working load. Input as a positive decimal fraction.

YS = minimum distance from the bottom fiber to the center of gravity of the prestressing steel (in.)

AS = area of prestressing steel (in.^{2})

FØ = initial prestressing force (lbs)

<u>Section Dimensions and Properties</u>

H = depth of precast section (in.)

HMAX(HMIN) = maximum (minimum) depth of precast section (in.)

BT = top flange width (in.)

BTMAX(BTMIN) = maximum (minimum) top flange width (in.)

BTHMAX = maximum top flange width to total depth ratio

BBMAX(BBMIN) = maximum (minimum) bottom flange width (in.)

BBHMAX = maximum bottom flange width to total depth ratio

BBHMIN = minimum width to depth ratio. Applies only to rectangular sections

BW = web thickness (in.)

BWMAX(BWMIN) = maximum (minimum) web thickness (in.)

BWHMIN = minimum web thickness to total depth ratio

TT = top flange thickness (in.)

TTMAX(TTMIN) = maximum (minimum) top flange thickness (in.)

TTMAX(TTTMIN) = maximum (minimum) top flange thickness to width ratio

TB = bottom flange thickness (in.)

TBMAX(TBMIN) = maximum (minimum) bottom flange thickness (in.)

TBBMAX(TBBMIN) = maximum (minimum) bottom flange thickness to width ratio

CB(CT) = bottom (top) flange taper (in.)

BS = effective slab width (in.)

TS = slab thickness (in.)

GS = slab groove (in.) (see Figure 16)

EPCST = modulus of elasticity of precast concrete (psi)

ESLAB = modulus of elasticity of cast-in-place slab (psi)

GAMMA = specific weight of precast concrete section (pcf)

CRF = creep factor--the factor by which the modulus of elasticity is reduced due to creep and shrinkage for deflection computations.

Program Modes

There are two general program modes for user consideration: the design mode which computes a girder design based on user input specification; and the check mode which computes properties, profile limits, and stresses for a given girder. Input under the two modes is formatted in exactly the same way, but there are certain parameters which apply under one mode but are meaningless under the other. For example, it would be pointless to specify BT, BB, FØ, or AS under the design mode since these parameters are all computed by the program and inputting their values makes no difference as to their final values. Under the check mode the four parameters (BT, BB, FØ, and AS) which were meaningless under the design mode are essential to the formulation of the problem mode while section dimension restraint parameters such as HMAX or BBHMAX would be meaningless. It should be noted that the specification of a meaningless parameter is ignored by the program, but a failure to specify a required parameter will cause program termination.

There is one case where a user may wish to specify AS = 0.0 under the check mode. Since the check mode is based on a net/transformed section it is possible to "trick" the program into analyzing a section based on its gross section properties by specifying a zero area of steel. Every computation in this case is carried out as usual with the area of steel included as zero except that ultimate strength computations are by-passed and the ultimate flexural capacity is out as zero.

Loading Specifications

The program is designed to handle both static and moving loads. The user may specify up to five separate loading combinations which may include any number and combination of uniform, uniform segment, and concentrated loadings, combined with any AASHO standard truck or equivalent lane loadings. The moments and shears for each loading combination are evaluated and printed out, while a maximum shear and moment envelope is determined from all loading combinations and used for design and checking computations.

Input

Loading input consists of a series of loading cards in a logical sequence described by the flow chart of Figure 19. Each card contains coded information formatted as described in Appendix C.

The user is required to begin each loading combination with a card containing the combination number and the total number of loadings in the loading combination. Combination numbers must be numbered consecutively starting with one until all loading combinations have been read in, then the last card must specify the loading combination number as zero, signaling the program to terminate reading loading cards. The number of loadings specified must reflect each separate type of loading included under that

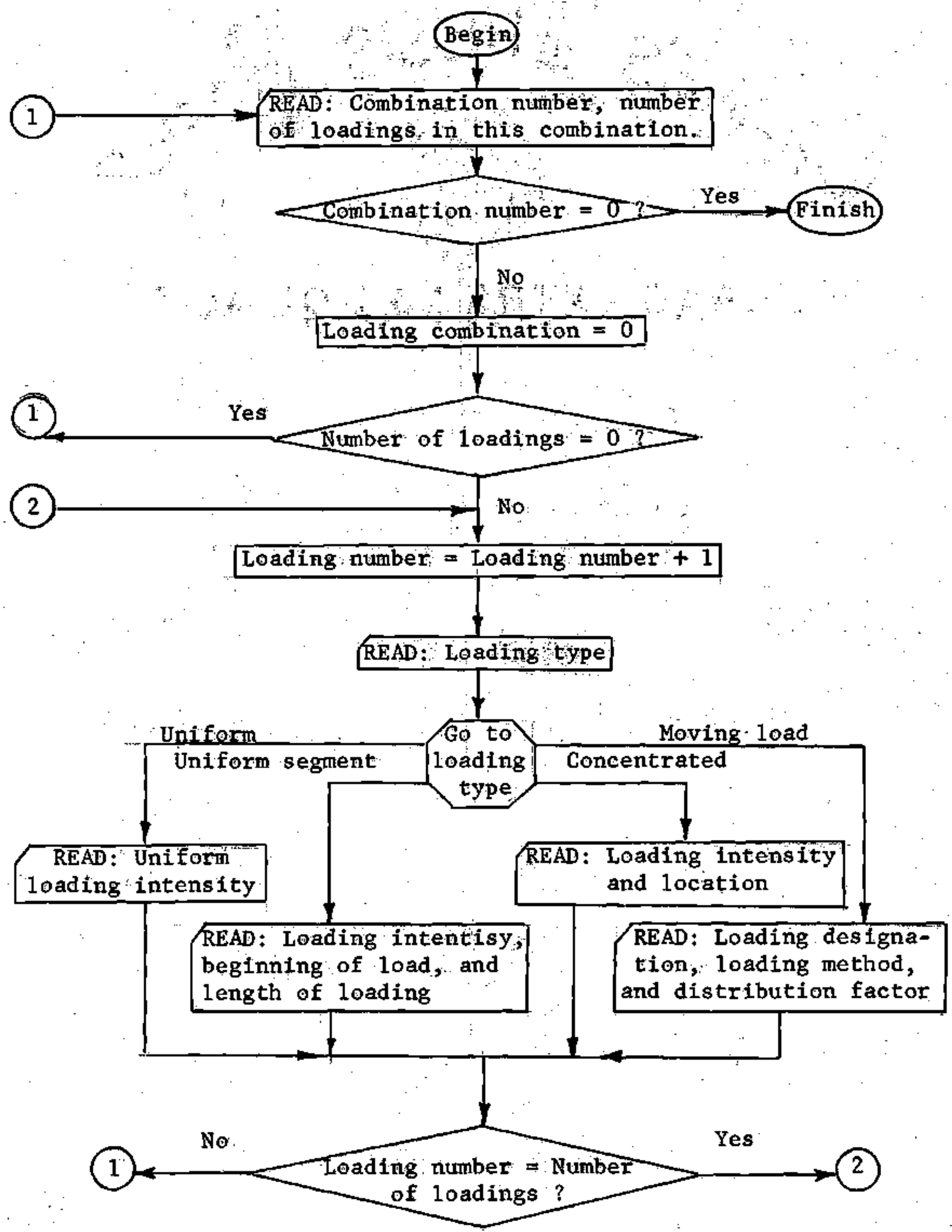


Figure 19. Loading Input Flow Chart

combination. For instance, if the user wished to include a uniform load, two concentrated loads, and a moving load, he would specify the number of loadings as four.

The loading combination number and number of loadings card is followed by a series of two cards each corresponding to each of the number of loadings specified. The first card in the dual card series notes the loading type to be described by the second card. The loading type may be one of the following four: uniform, uniform segment, concentrated, or moving load. The description of a uniform loading involves only the specification of a uniform loading intensity in kips per foot. A uniform segment loading must be described by not only specifying intensity (kip/ft) but also the location of the beginning of the load from the left support (ft) and the total length of the load from its beginning (ft). Concentrated loads are specified by intensity (kips) and location from the left support (ft). The description of a moving load involves options not involved in the previous three loading types. The user may specify any of the five (H20-44, H15-44, H10-44, HS20-44, HS15-44) AASHO truck loading designations and specify whether it be analyzed as a standard truck load or equivalent lane load. The user is also required to specify a live load distribution coefficient which accounts for the proportion of a single lane loading to be resisted by a single girder.

Coded Specifications

The loading specifications are input as either integers or real numbers. In most cases this type of input lends itself to the parameter being input, but in certain cases coded parameters must be used. The parameters are coded as follows:

Loading type specifications

Uniform	= 1
Uniform Segment	= 2
Concentrated	= 3
Moving Loads	= 4

AASHO moving load designations

H20-44	= 1
H15-44	= 2
H10-44	= 3
HS20-44	= 4
HS15-44	= 5

Loading method specifications

Standard truck loads	= 1
Equivalent land loads	= 2

Specification of Zero Loading

As seen from the flow chart in Figure 19, the program checks to see if the user specified the number of loadings under a certain combination as zero. It may seem pointless for the user to specify a loading condition under which there are no loadings, but this method is used to have the program print out the beam camber at transfer. Beam deflections at NSEG + 1 points along the span are computed and output for each loading combination specified by the user. So, if the user specifies no loading under a certain combination, the only forces acting on the beam are the prestress force and the dead weight of the beam. When zero is specified for the number of loadings, a flag is set which causes deflection computations to be based on the initial prestress force, a creep factor of one, and, in the case of computations under the check mode, a net section.

Example Problem

The following is an example of the input required for the design of a typical prestressed concrete highway bridge girder conforming to AASHO specifications.

Problem Definition

A design is required for an I beam spanning 75 feet. The beam is to be post-tensioned by a parabolic, bonded tendon, and the center of gravity of the steel is to be kept at a minimum of three inches from the bottom fiber of the beam. The slab is to be erected with shoring supporting the weight of the cast-in-place concrete. The effective width of the slab is 60 inches, its thickness is 5.5 inches, and the beam is set into the slab 2 inches. An asphalt surface topping will be applied weighing 100 plf. A highway standard will be erected at 27.3 feet from the left girder support contributing a 3.6 kip vertical load on the girder. The girder must withstand a HS20-44 standard truck load and, since there are 1.5 girders per lane, the distribution factor is 0.75. Because of clearance limitations, a 40 inch deep beam is desired, but the depth may increase to a maximum of 48 inches if required. Both the top and bottom flange width to total depth ratios are to be limited to a maximum of 0.75 because of forming considerations while both the top and bottom flange thickness to width ratios are to be held within the range of from 0.10 to 0.50. The 28 day concrete strength is assumed to be 4000 psi and the ultimate tendon strength is 270,000 psi. The modulus of elasticity of the precast girder is 5500 psi and that for the slab is 4500 psi. It is desired to know the camber of the beam at transfer, the deflected position of the beam under the influence of the beam, slab and asphalt weight only, and the deflected position under full design loads. A creep factor of 2.5 has been determined to be adequate for beams of this loading and type for this area.

Input

The following is a record, card by card, of the formatted input required for the problem previously stated. The exact format of the input is detailed in Appendix C and is presented here without regard for the exact position required on the card. Some parameters, such as the ultimate tendon strength, are left unspecified because their default values (see Appendix D) are the values required by the design.

```
TITLE CARD — PRINTED OUT AT BEGINNING OF OUTPUT
$SECT
DES = 1, COMP = 1, TYPE = 1, SUP = 1, TENS = 2, NESEG = 10,
LEN = 75.0, BOND = 1,
$END
    1       3
    1
   0.10
    3
   3.6             27.3
    4
    4       1          0.75
    2       2
    1
   0.10
    3
   3.6             27.3
    3       0
    0
$PROP
  H = 40.0, HMAX = 48.0, BBHMAX = 0.75, BTHMAX = 0.75, TTTMAX = 0.5,
  TTTMIN = 0.10, TBBMAX = 0.5, TBBMIN = 0.10, FCP = 4000.0, EPCST =
  5500.0, ESLAB = 4500.0, YS = 2.5,
$END 1
```

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

The programmed technique developed in this thesis provides the user with an economical and efficient prestressed concrete section. But the advantage of a programmed design technique does not end with a means of providing an adequate girder design to meet the input requirements; it can provide the means of acquiring experience of alternative designs; of the effect of different shaped sections, and so on, in a short space of time. The programmed design enables the judgment and engineering sense of the designer to be better informed than it can possible be without it.

Recommendations for Further Program Enhancements

The design program was developed to design any simple spanned prestressed concrete girder with special consideration being given to the requirements of highway bridge girder design. The obvious extension to this program would be the inclusion of continuous spans. While the design technique used in the present program would not easily lend itself to continuous span design, it would, with proper modification, be applicable to the design of a simple spanned girder with some constant end moments.

Another possible program extension would be the inclusion of girders with a variable section. This extension may not be as difficult as the inclusion of continuous spans since the check mode already allows for variable section properties generated by the variable tendon eccentricity and the net/transformed section. The problem involved here is the determination of the critical section for design purposes. This is one reason why design is based on the gross section and not a net/transformed section.

Because of the slenderness and flexibility of typical prestressed concrete girders, they are susceptible to a resonance condition with their natural modes of vibration. Perhaps a subroutine to examine the natural frequency of the designed girder would enhance the usefulness of the program.

The program now computes the deflected shape of the girder for each loading condition imposed. The deflections are output for the information of the user, but no attempt has been made to design based on meeting deflection requirements. A possible extension, therefore, would be to recycle the design each time a girder did not meet a set of deflection requirements and make logical adjustments until the requirements were satisfied.

In the program's present form, shears are computed and output for the user's information. No attempt has been made to either design or check for shear requirements. A subroutine could be written to check the girder's shear capacity and this information output under the check mode; while under the design mode a shear check could be made, and if violated, the girder redesigned.

APPENDIX A

COMPUTER PROGRAMS

MAIN: The main control program of the system; reads input information, makes data checks and assigns various parameters default values if required, makes calls to appropriate sub-programs, and outputs computed values.

CALLS: LOADS, CHECK, DESIGN, RECT, STRESS, PROFIL, ECCEN, COMPOS, RECTC, STRCOM, PROFIC, DEFLN

LOADS: The control program for determining the maximum shear and moment envelope for various combinations of static and moving loads; reads loading information and outputs maximum moments and shears for a user specified number of points along the span.

CALLS: MOMSHR

CALLED BY: MAIN

MOMSHR: Computes shears and moments at a user specified number of points along the girder for each of the specified loadings.

CALLS: MOMLIV, COMBIN

CALLED BY: LOADS

MOMLIV: Generates the maximum moment envelope for standard AASHO truck loadings.

CALLED BY: MOMSHR

COMBIN: Combines the moments and shears for the individual loadings computed by MOMSHR into the total moment and shear diagrams for each loadings combination.

CALLED BY: MOMSHR

CHECK: The control program for the check mode of the system; reads input information, makes data checks and assigns various parameters default values if required, calls appropriate sub-programs and outputs computed data for a net/transformed section.

CALLS: SECPRP, ECCEN, TRANSF, PROFIL, STRX, PROFIC, STRCX, ULTSTR, DEFLTR

CALLED BY: MAIN

SECPRP: Computes section properties based on the gross area of I, T, or rectangular section with either composite or non-composite action.

CALLED BY: CHECK, COMPOS, RECTC

ECCEN: Generates tendon profile from midspan eccentricity and user specified tendon description.

CALLED BY: CHECK, MAIN

TRANSF: Computes section properties at a user specified number of points along the span based on both the net and transformed concrete areas.

CALLED BY: CHECK

PROFIL: Generates band of limiting tendon eccentricities for a given prestress force and computes the maximum and minimum prestress force with corresponding eccentricities for non-composite sections.

CALLED BY: CHECK, MAIN

STRX: Computes initial and final stresses in the top and bottom fibers for non-composite sections at a user specified number of points along the length of the beam.

CALLED BY: CHECK

PROFIC: Same as PROFIL except called for composite sections.

CALLED BY: CHECK, MAIN

STRCX: Same as STRX except called for composite sections.

CALLED BY: CHECK

ULTSTR: Computes the ultimate flexural capacity of a general composite or non-composite section.

CALLED BY: CHECK, DESIGN, COMPOS, RECT, RECTC

DEFLTR: Computes deflections based on the net/transformed properties of the section at a user specified number of points along the span.

CALLED BY: CHECK

DESIGN: Designs I or T non-composite sections by working stress theory subject to all user specified dimension constraints and ultimate strength requirements.

CALLS: ULTSTR

CALLED BY: MAIN

RECT: Same as DESIGN except applicable to rectangular sections only.

CALLS: ULTSTR

CALLED BY: MAIN

STRESS: Computes initial and final stresses in the top and bottom fibers of non-composite sections at midspan.

CALLED BY: MAIN

COMPOS: Same as DESIGN except called for composite sections.

CALLS: SECPRP, ULTSTR

CALLED BY: MAIN

RECTC: Same as COMPOS except applicable to rectangular sections only.

CALLS: SECPRP, ULTSTR

CALLED BY: MAIN

STRCOM: Same as STRESS except called for composite sections.

CALLED BY: MAIN

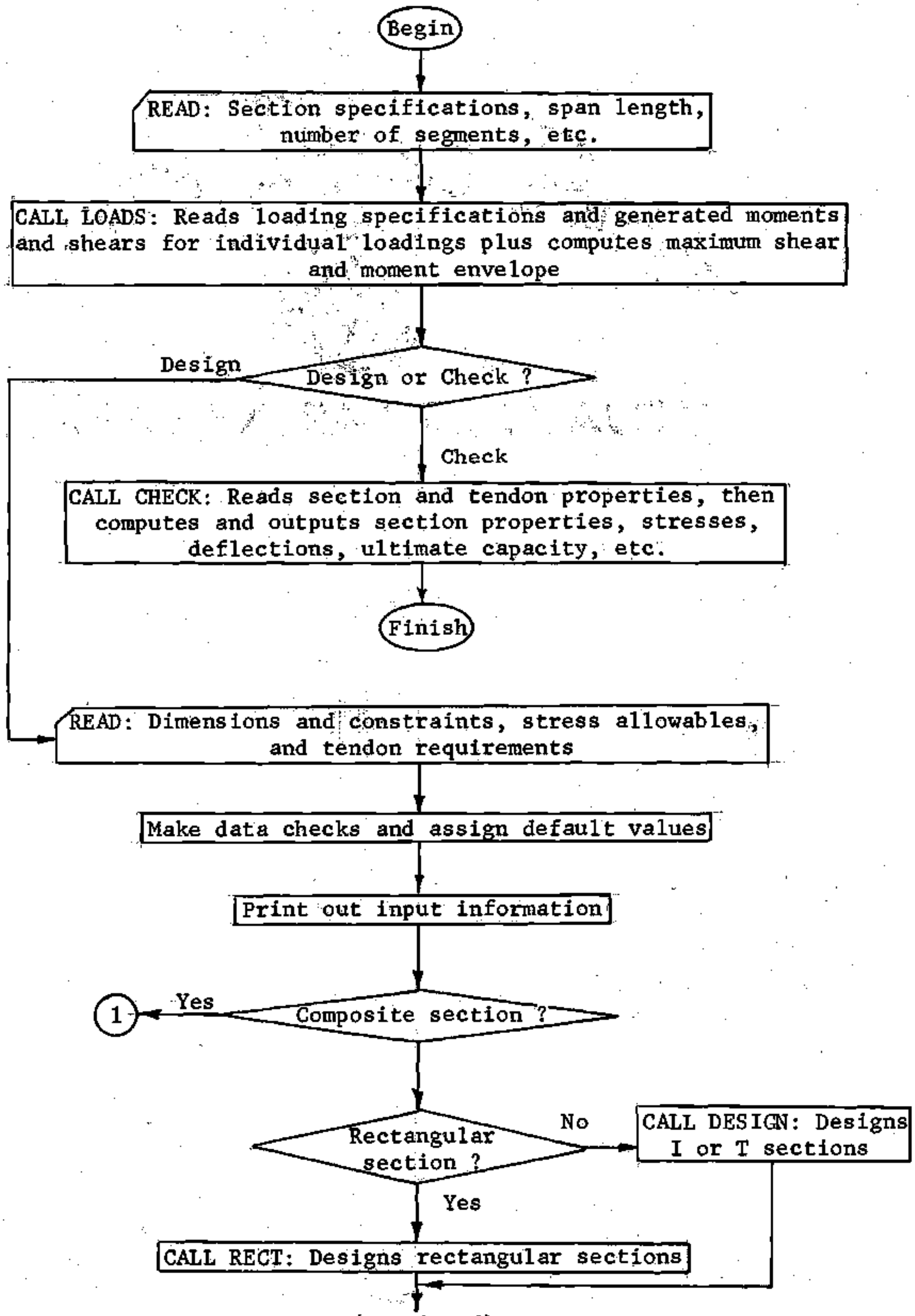
DEFLN: Computes deflections based on the gross properties of the section at a user specified number of points along the span.

CALLED BY: MAIN

APPENDIX B

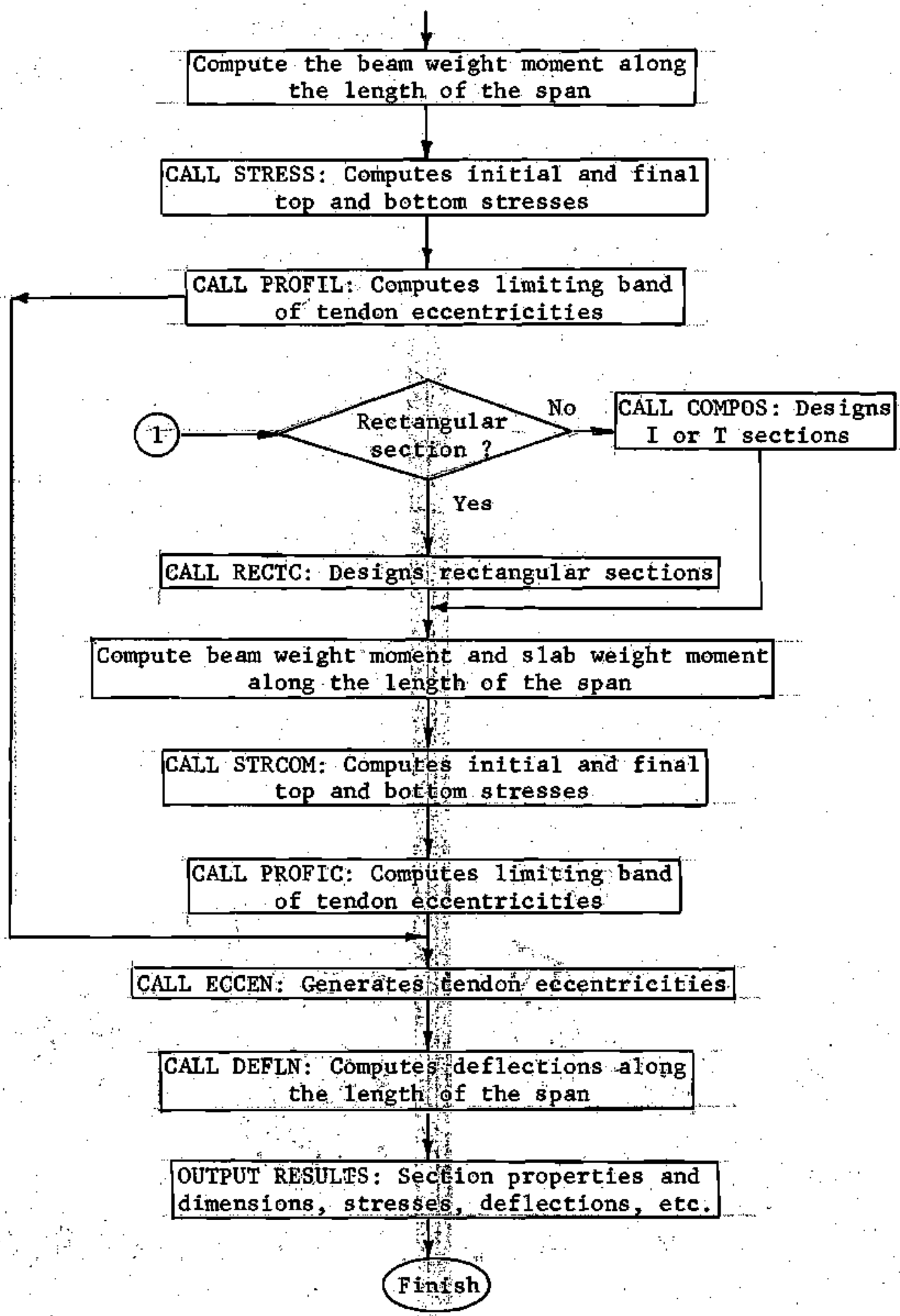
FLOW CHARTS

MAIN

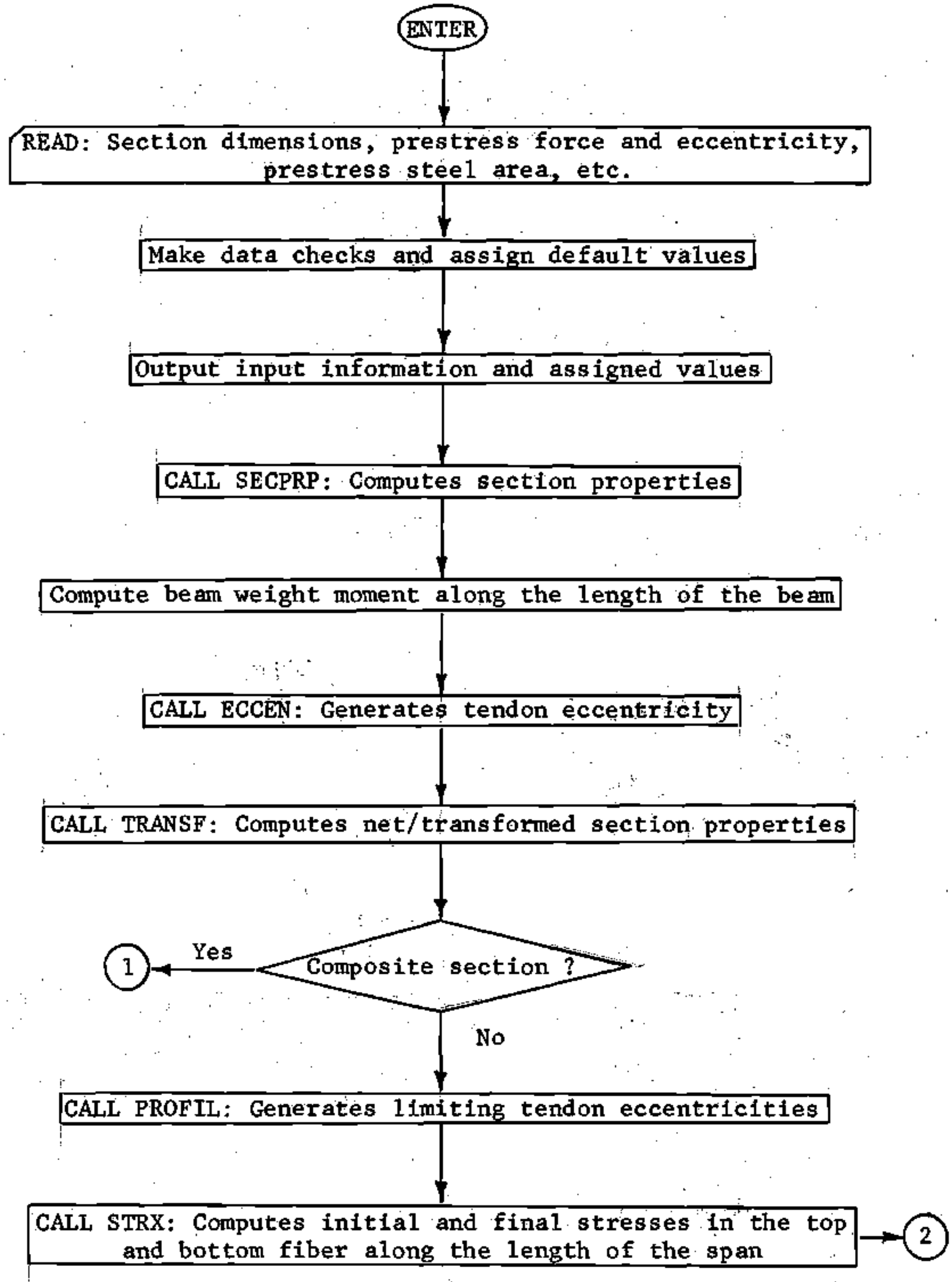


(continued)

MAIN (cont.)

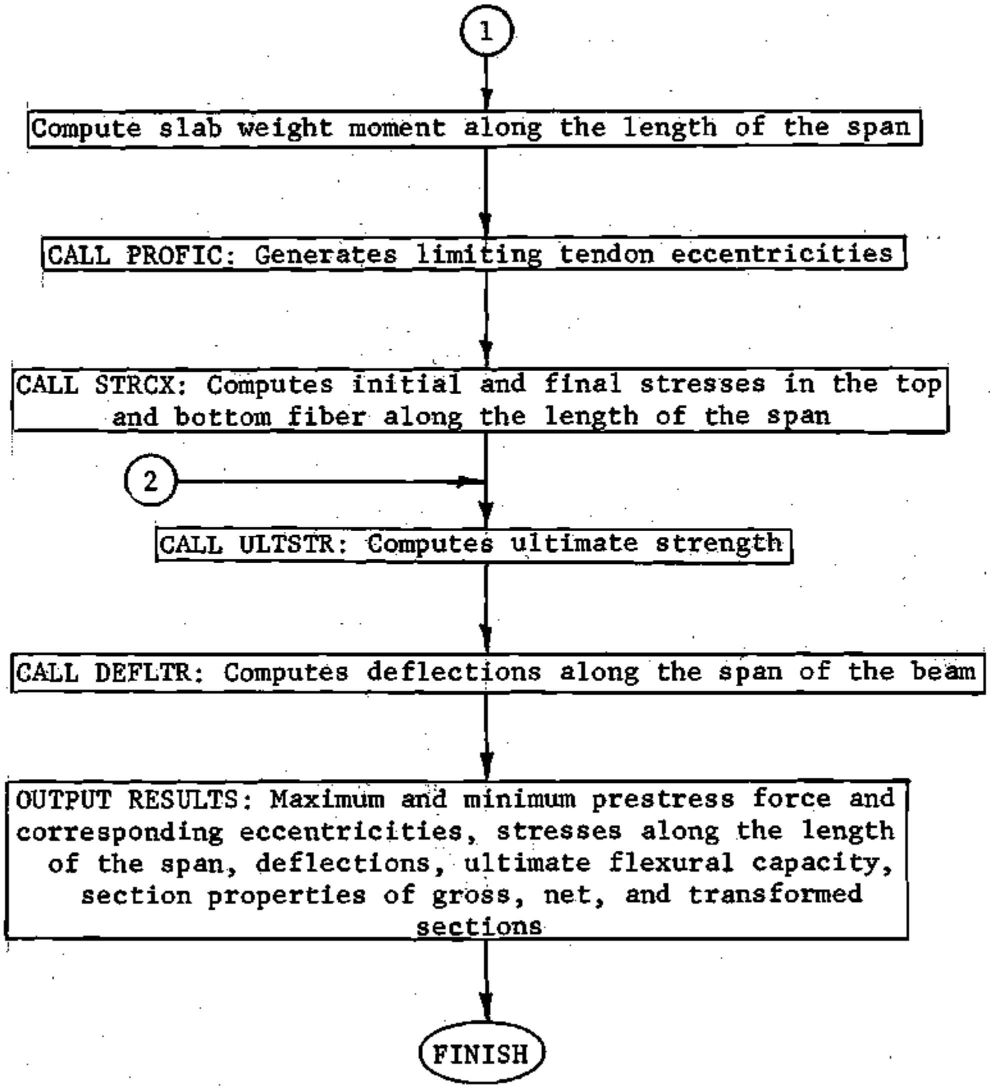

Compute the beam weight moment along the length of the span
CALL STRESS: Computes initial and final top and bottom stresses
CALL PROFIL: Computes limiting band of tendon eccentricities
1
Rectangular section ?
No
CALL COMPOS: Designs I or T sections
Yes
CALL RECTC: Designs rectangular sections
Compute beam weight moment and slab weight moment along the length of the span
CALL STRCOM: Computes initial and final top and bottom stresses
CALL PROFIC: Computes limiting band of tendon eccentricities
CALL ECCEN: Generates tendon eccentricities
CALL DEFLN: Computes deflections along the length of the span
OUTPUT RESULTS: Section properties and dimensions, stresses, deflections, etc.
Finish

CHECK

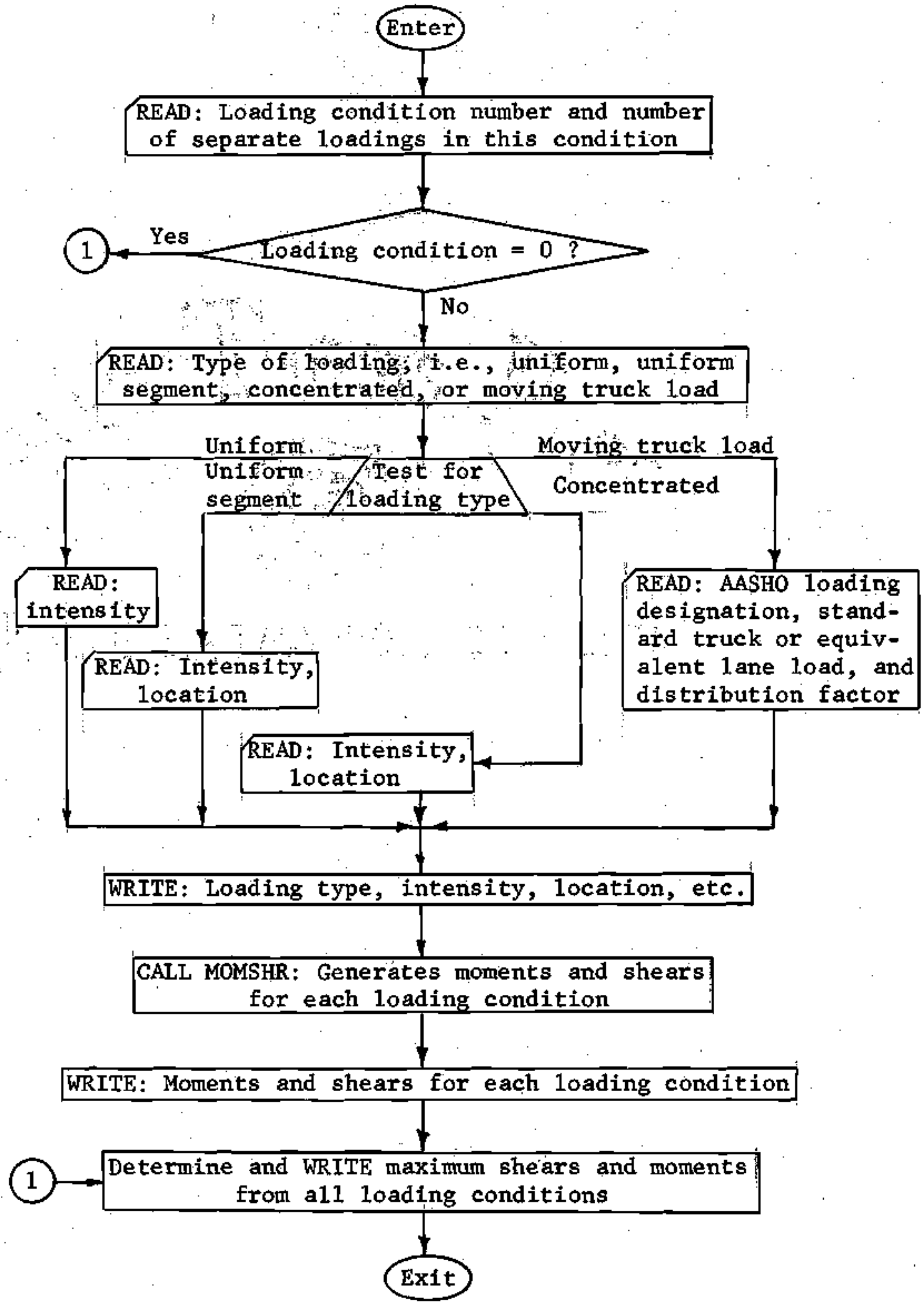


(continued)

CHECK (cont.)

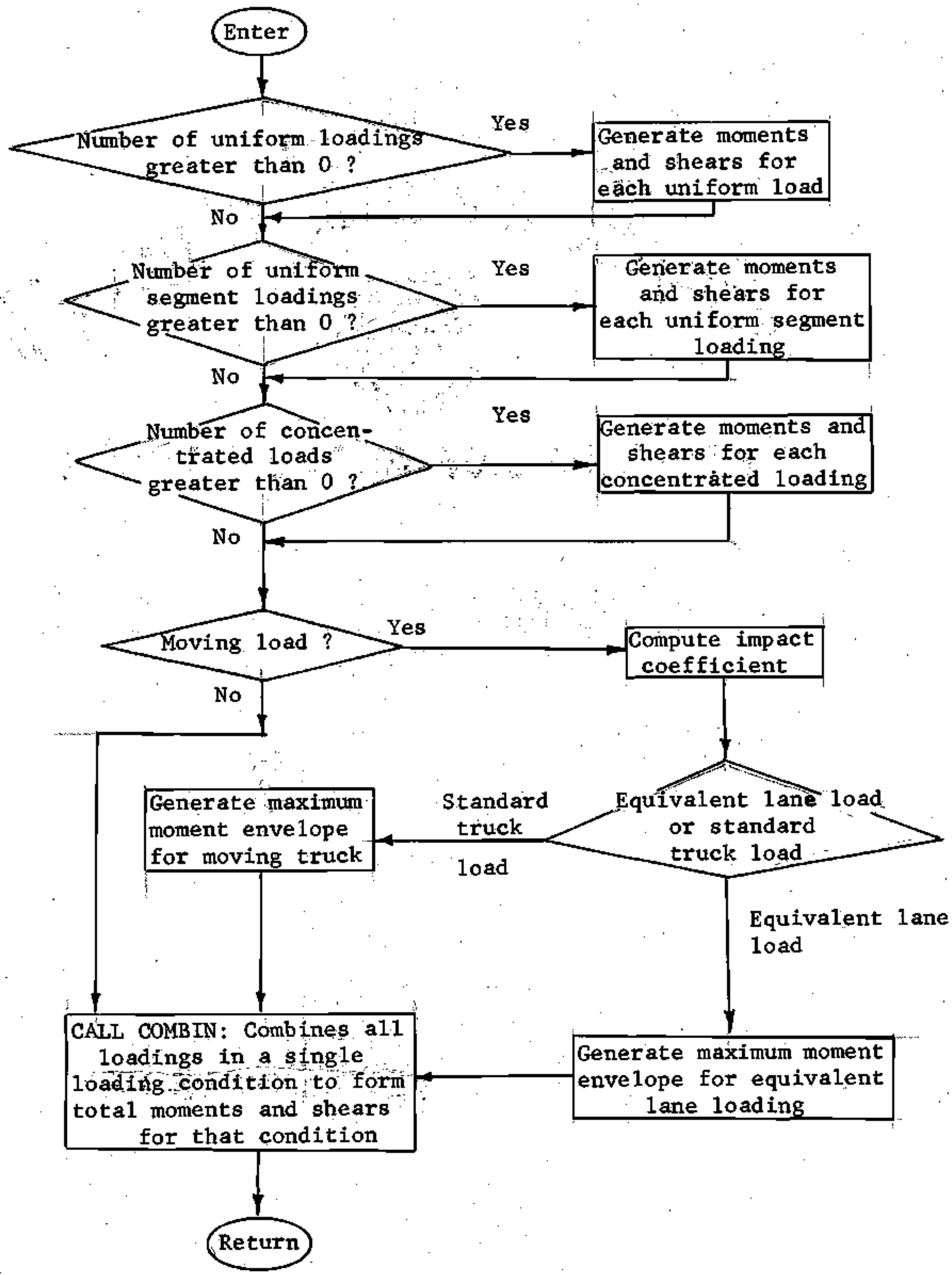

1
Compute slab weight moment along the length of the span
CALL PROFIC: Generates limiting tendon eccentricities
CALL STRCX: Computes initial and final stresses in the top and bottom fiber along the length of the span
2
CALL ULTSTR: Computes ultimate strength
CALL DEFLTR: Computes deflections along the span of the beam
OUTPUT RESULTS: Maximum and minimum prestress force and corresponding eccentricities, stresses along the length of the span, deflections, ultimate flexural capacity, section properties of gross, net, and transformed sections
FINISH

LOADS

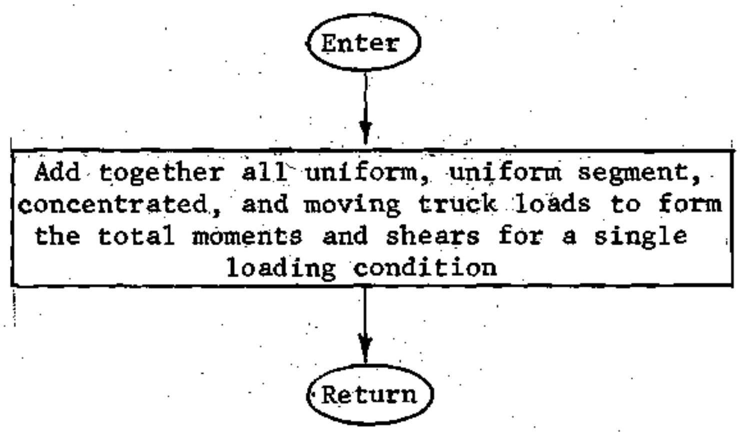


Enter
READ: Loading condition number and number of separate loadings in this condition
Loading condition = 0 ?
Yes
1
No
READ: Type of loading, i.e., uniform, uniform segment, concentrated, or moving truck load
Test for loading type
Uniform
Uniform segment
Moving truck load
Concentrated
READ: intensity
READ: Intensity, location
READ: Intensity, location
READ: AASHO loading designation, standard truck or equivalent lane load, and distribution factor
WRITE: Loading type, intensity, location, etc.
CALL MOMSHR: Generates moments and shears for each loading condition
WRITE: Moments and shears for each loading condition
1
Determine and WRITE maximum shears and moments from all loading conditions
Exit

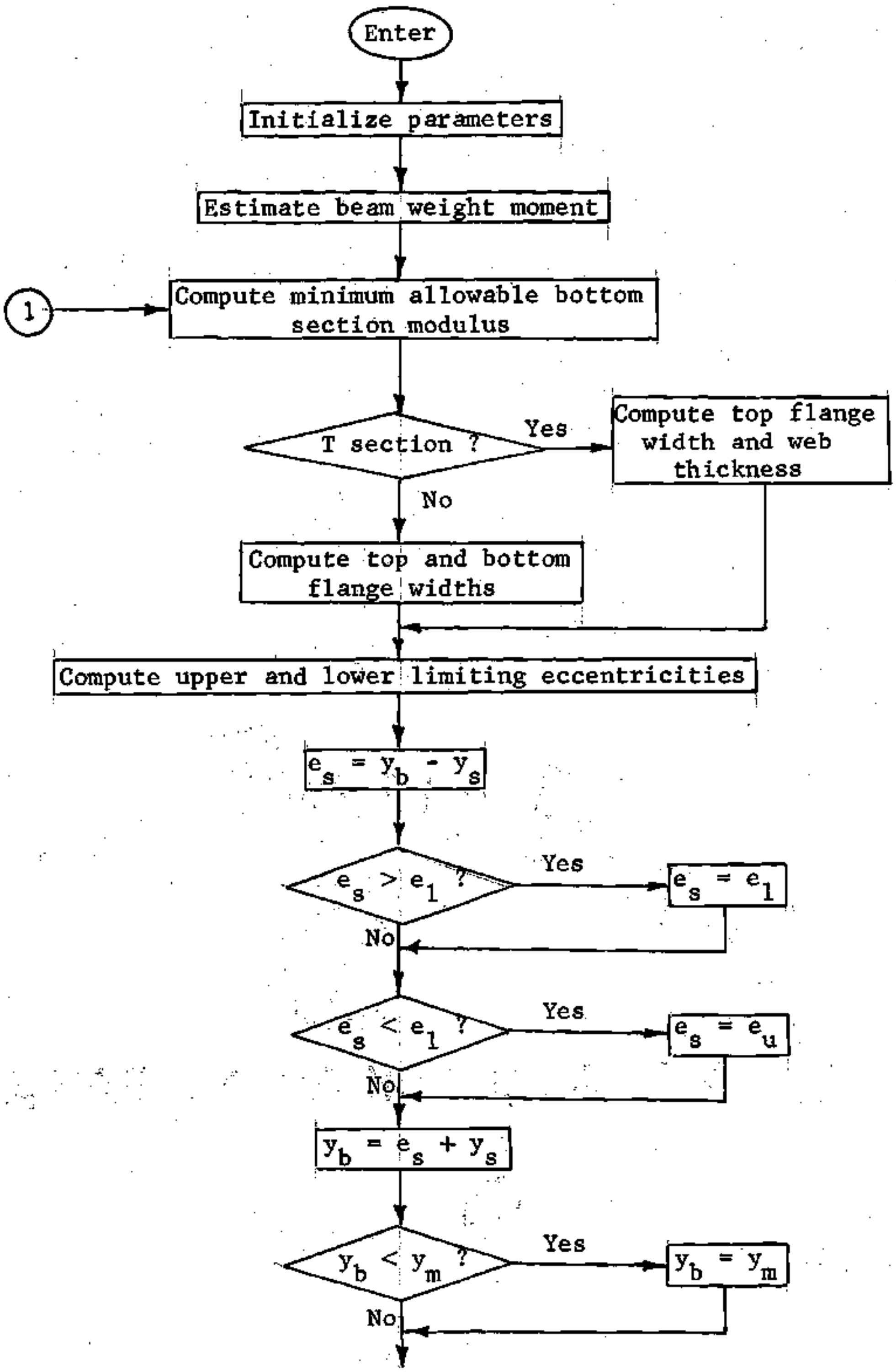
MOMSHR


Enter
Number of uniform loadings greater than 0 ?
Yes
Generate moments and shears for each uniform load
No
Number of uniform segment loadings greater than 0 ?
Yes
Generate moments and shears for each uniform segment loading
No
Number of concentrated loads greater than 0 ?
Yes
Generate moments and shears for each concentrated loading
No
Moving load ?
Yes
Compute impact coefficient
No
Generate maximum moment envelope for moving truck
Standard truck load
Equivalent lane load or standard truck load
Equivalent lane load
CALL COMBIN: Combines all loadings in a single loading condition to form total moments and shears for that condition
Generate maximum moment envelope for equivalent lane loading
Return

COMBIN

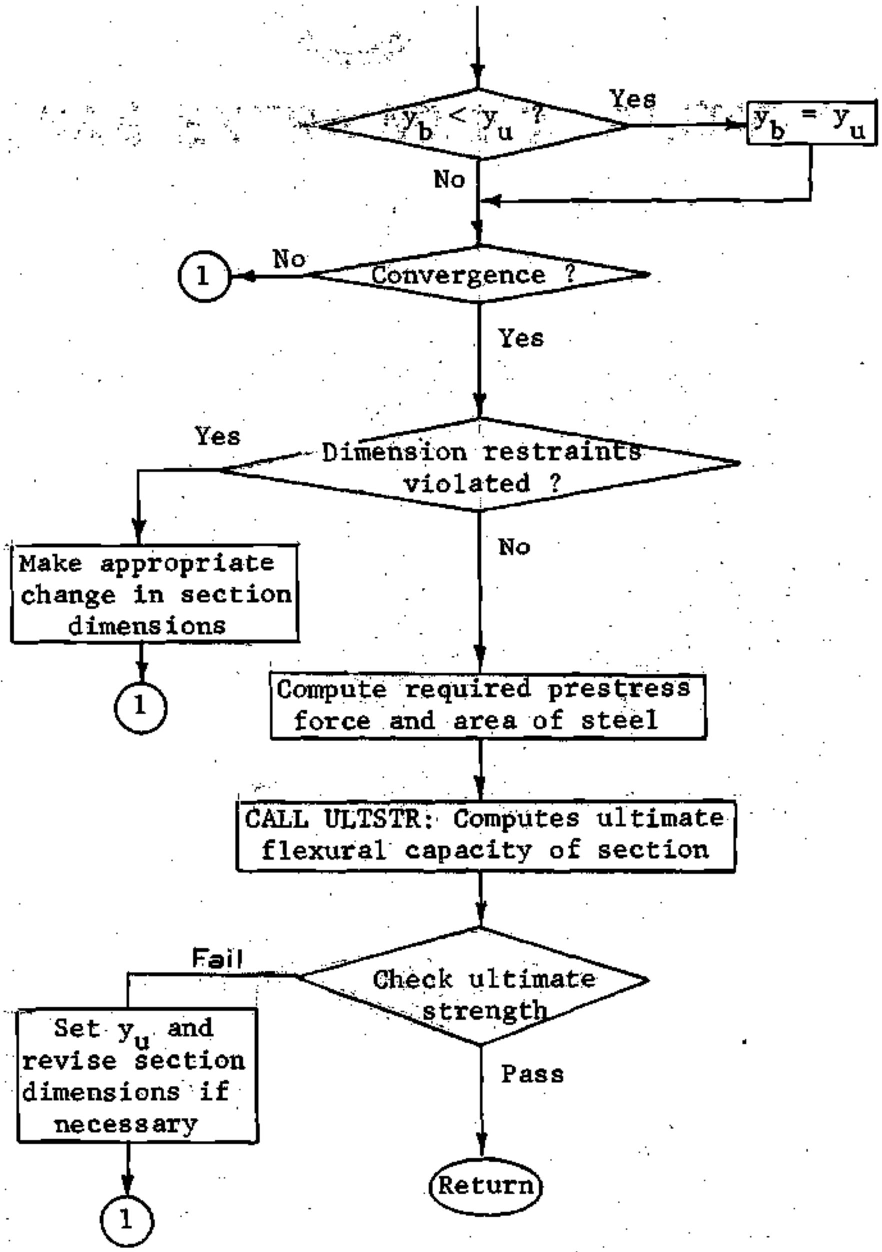

Enter
Add together all uniform, uniform segment, concentrated, and moving truck loads to form the total moments and shears for a single loading condition
Return

DESIGN



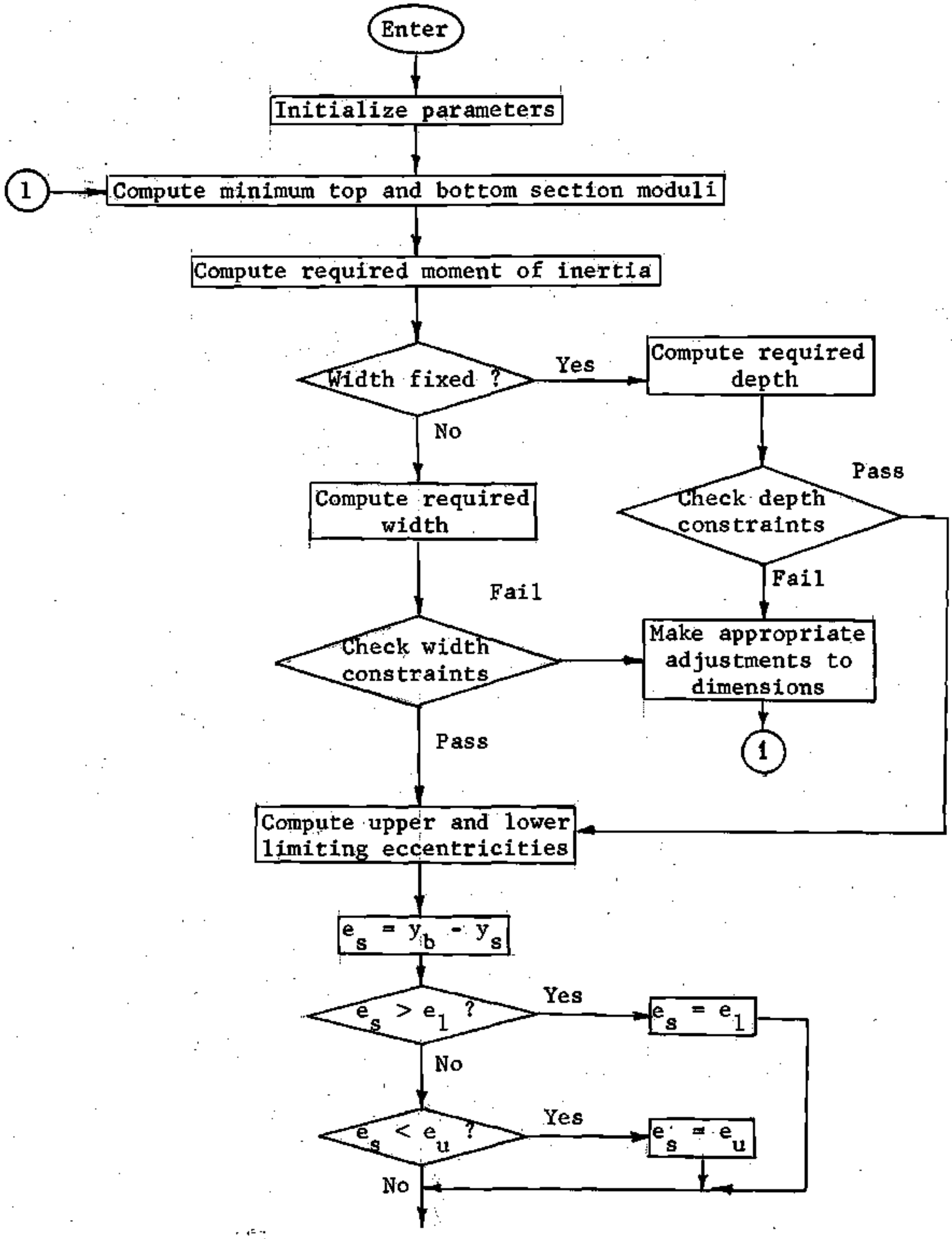
(continued)

DESIGN (cont.)



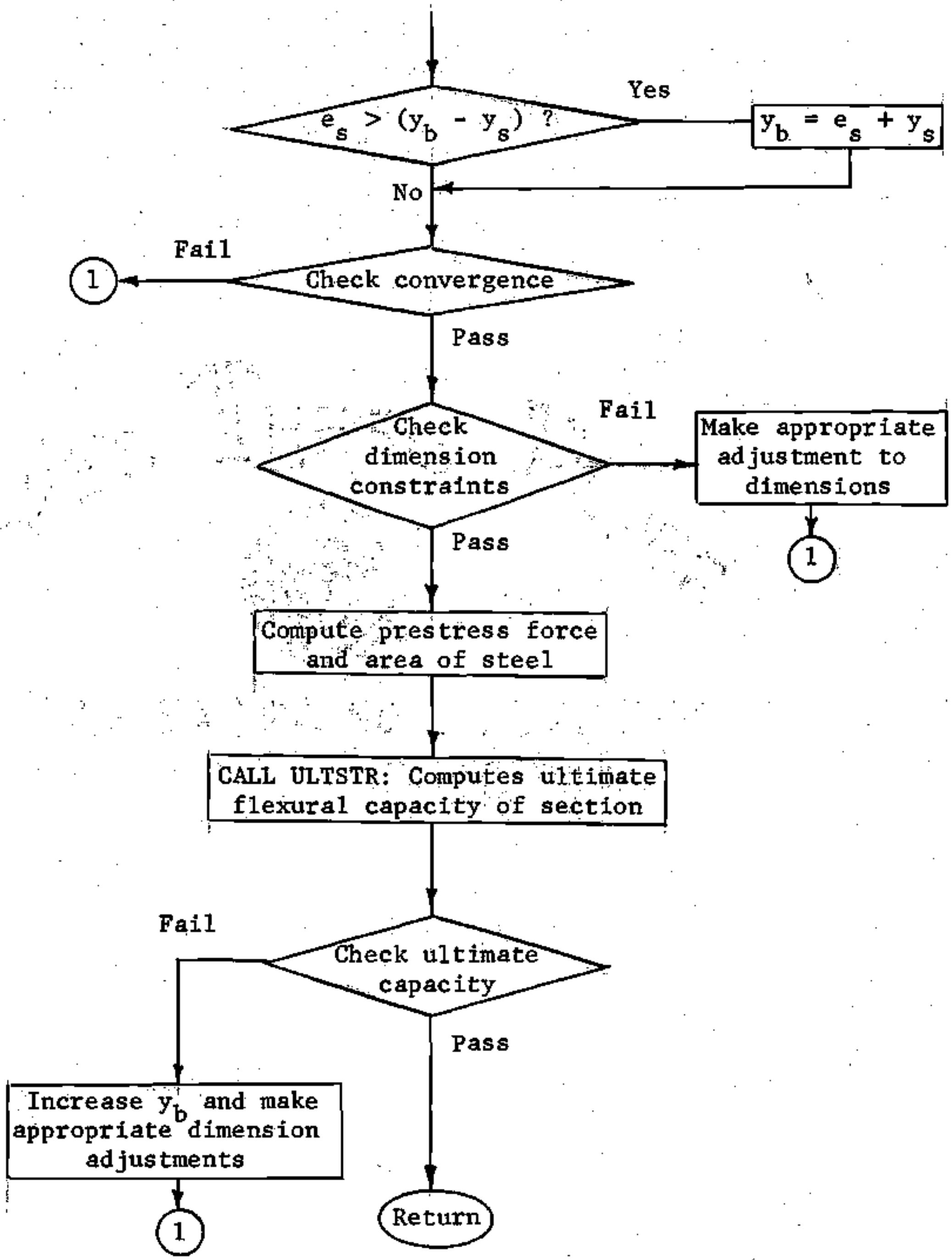
$y_b < y_u$?
Yes
$y_b = y_u$
No
No
1
Convergence ?
Yes
Yes
Dimension restraints violated ?
Make appropriate change in section dimensions
1
No
Compute required prestress force and area of steel
CALL ULTSTR: Computes ultimate flexural capacity of section
Fail
Check ultimate strength
Set y_u and revise section dimensions if necessary
1
Pass
Return

RECT

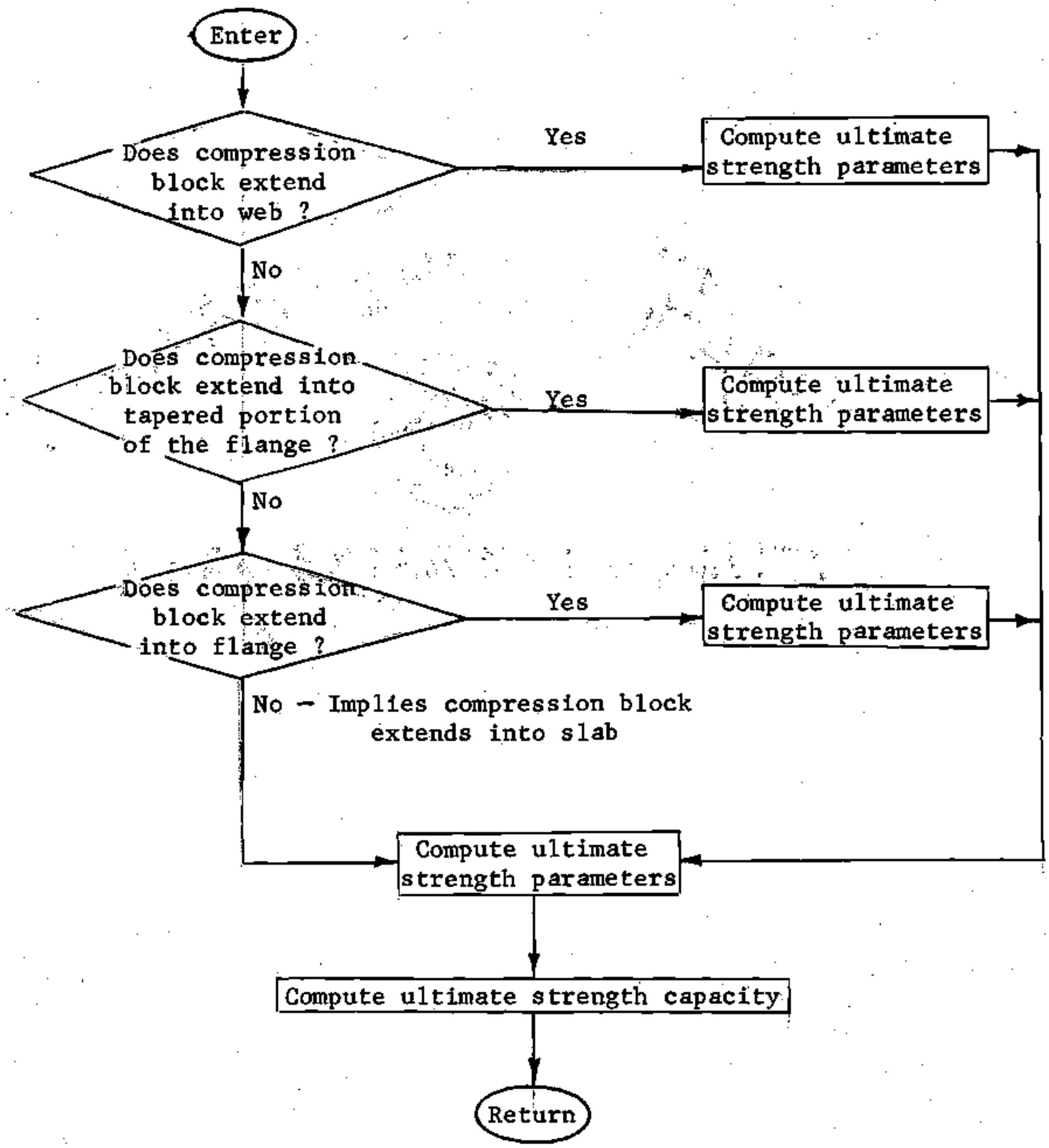


(continued)

RECT (cont.)

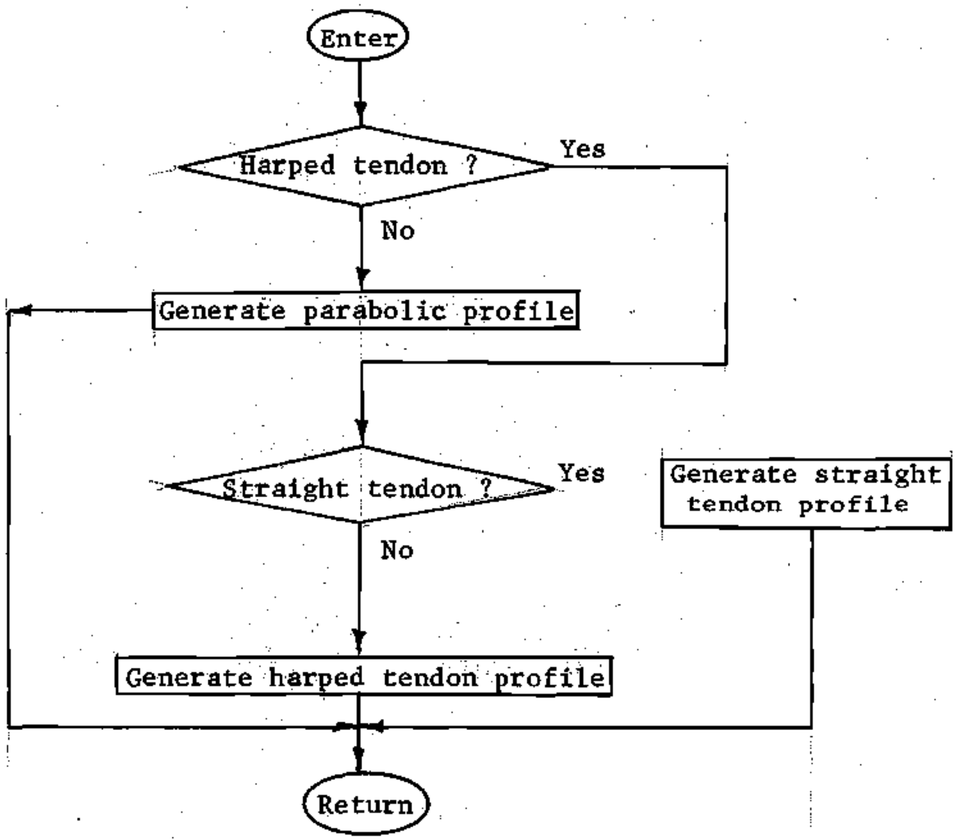

$e_s > (y_b - y_s)$?
Yes
$y_b = e_s + y_s$
No
Fail
Check convergence
Pass
Check dimension constraints
Fail
Make appropriate adjustment to dimensions
1
Pass
Compute prestress force and area of steel
CALL ULTSTR: Computes ultimate flexural capacity of section
Fail
Check ultimate capacity
Pass
Increase y_b and make appropriate dimension adjustments
1
Return

ULTSTR



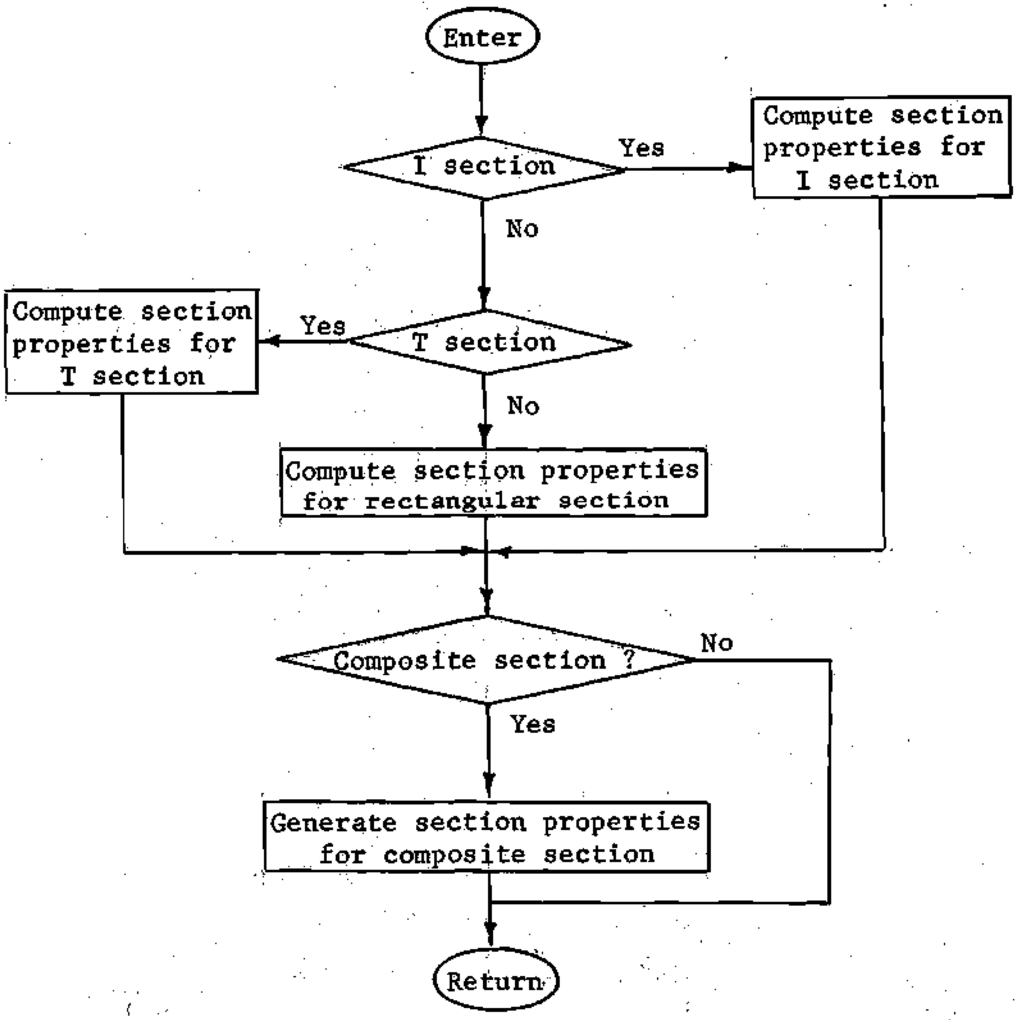
Enter
Does compression block extend into web ?
Yes
Compute ultimate strength parameters
No
Does compression block extend into tapered portion of the flange ?
Yes
Compute ultimate strength parameters
No
Does compression block extend into flange ?
Yes
Compute ultimate strength parameters
No — Implies compression block extends into slab
Compute ultimate strength parameters
Compute ultimate strength capacity
Return

ECCEN

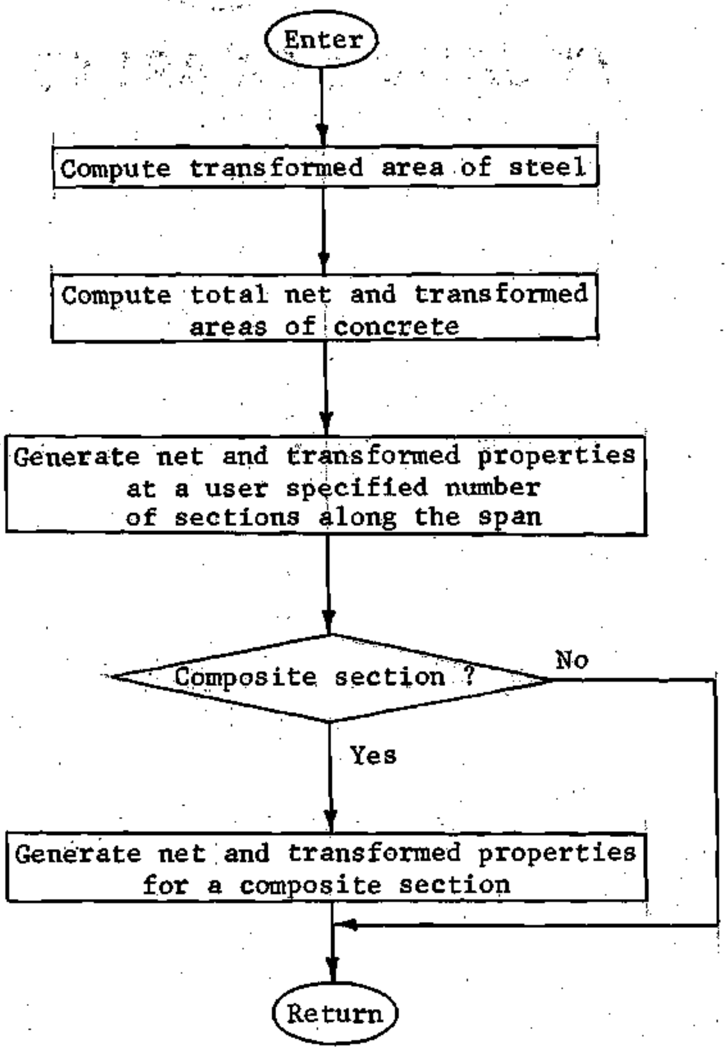


Enter
Harped tendon ?
Yes
No
Generate parabolic profile
Straight tendon ?
Yes
Generate straight tendon profile
No
Generate harped tendon profile
Return

SECPRP

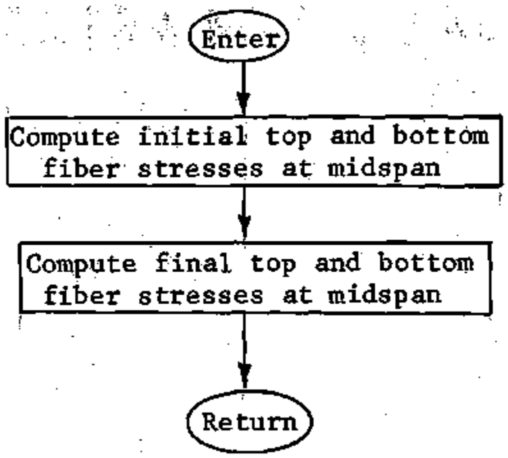

Enter
I section
Yes
No
Compute section properties for I section
T section
Yes
No
Compute section properties for T section
Compute section properties for rectangular section
Composite section ?
No
Yes
Generate section properties for composite section
Return

TRANSF

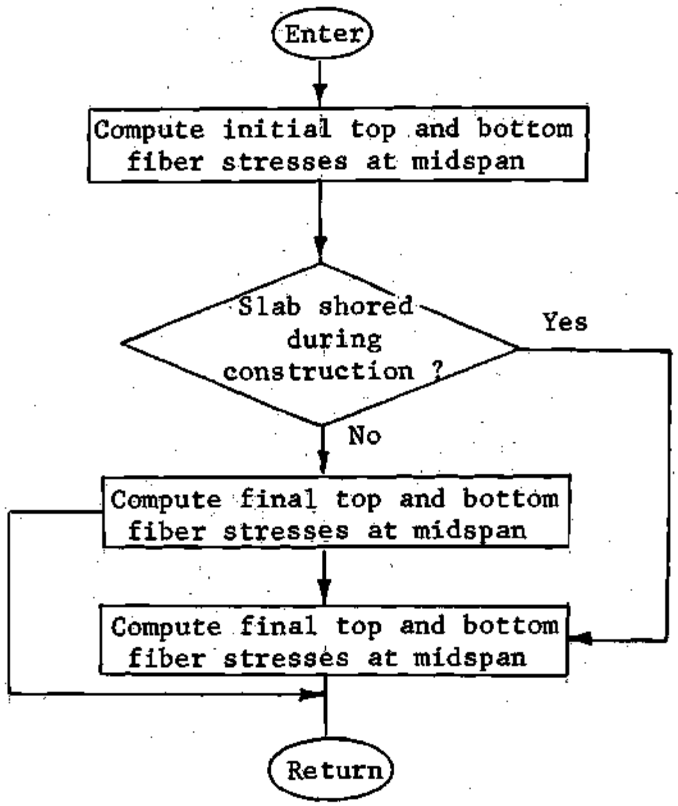


Enter
Compute transformed area of steel
Compute total net and transformed areas of concrete
Generate net and transformed properties at a user specified number of sections along the span
Composite section ?
No
Yes
Generate net and transformed properties for a composite section
Return

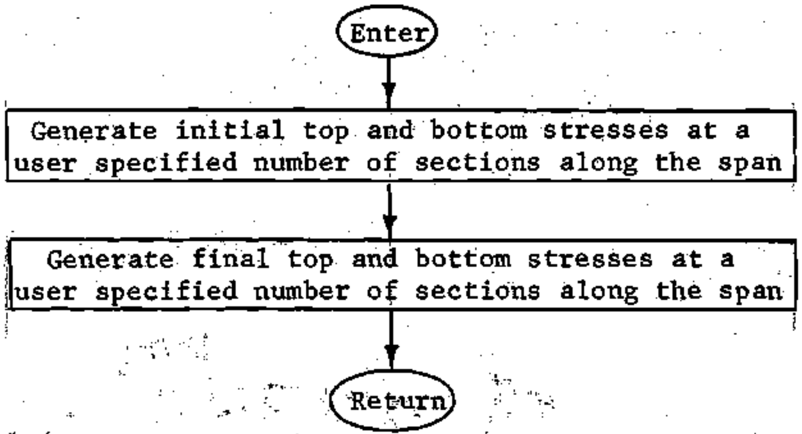
STRESS



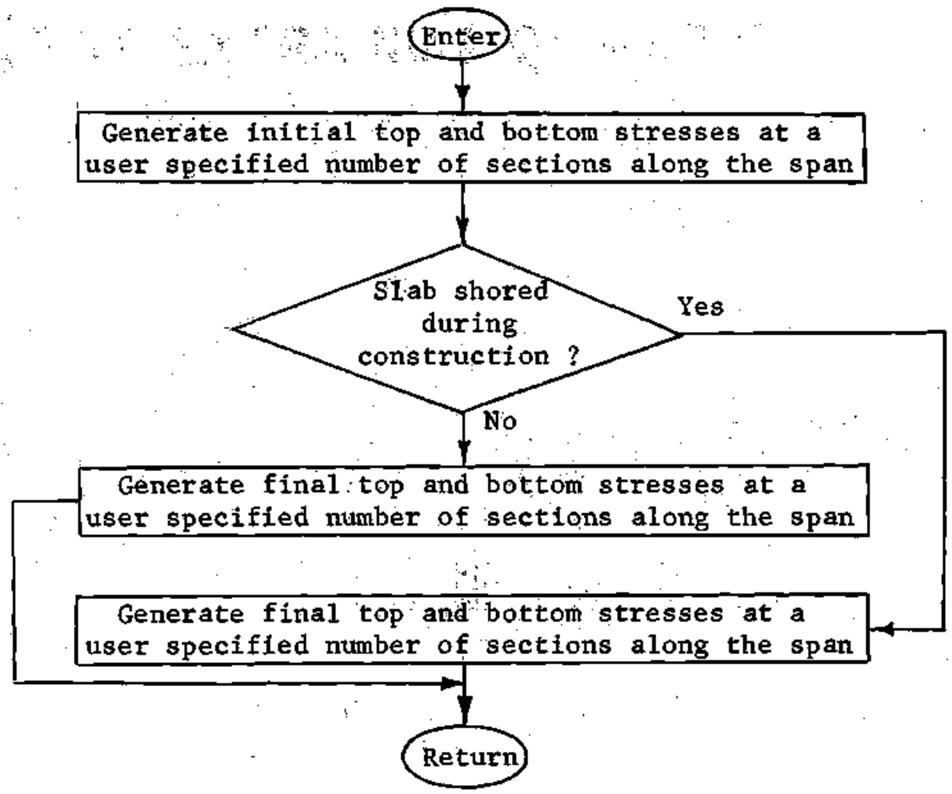
STRCOM



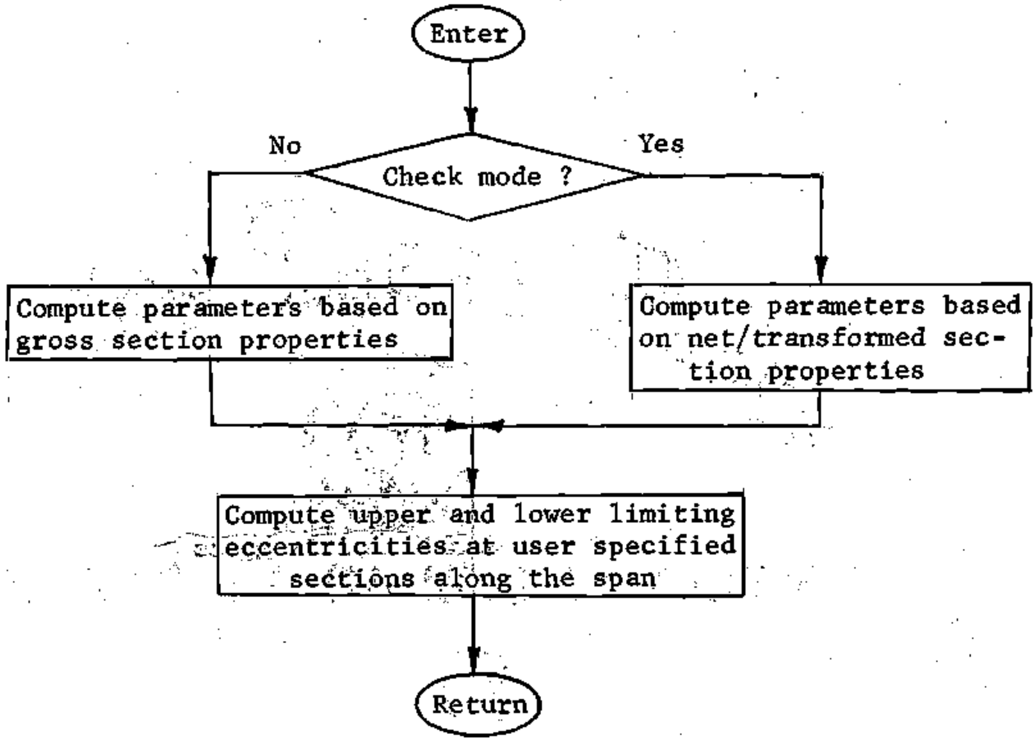
STRX


Enter
Generate initial top and bottom stresses at a user specified number of sections along the span
Generate final top and bottom stresses at a user specified number of sections along the span
Return

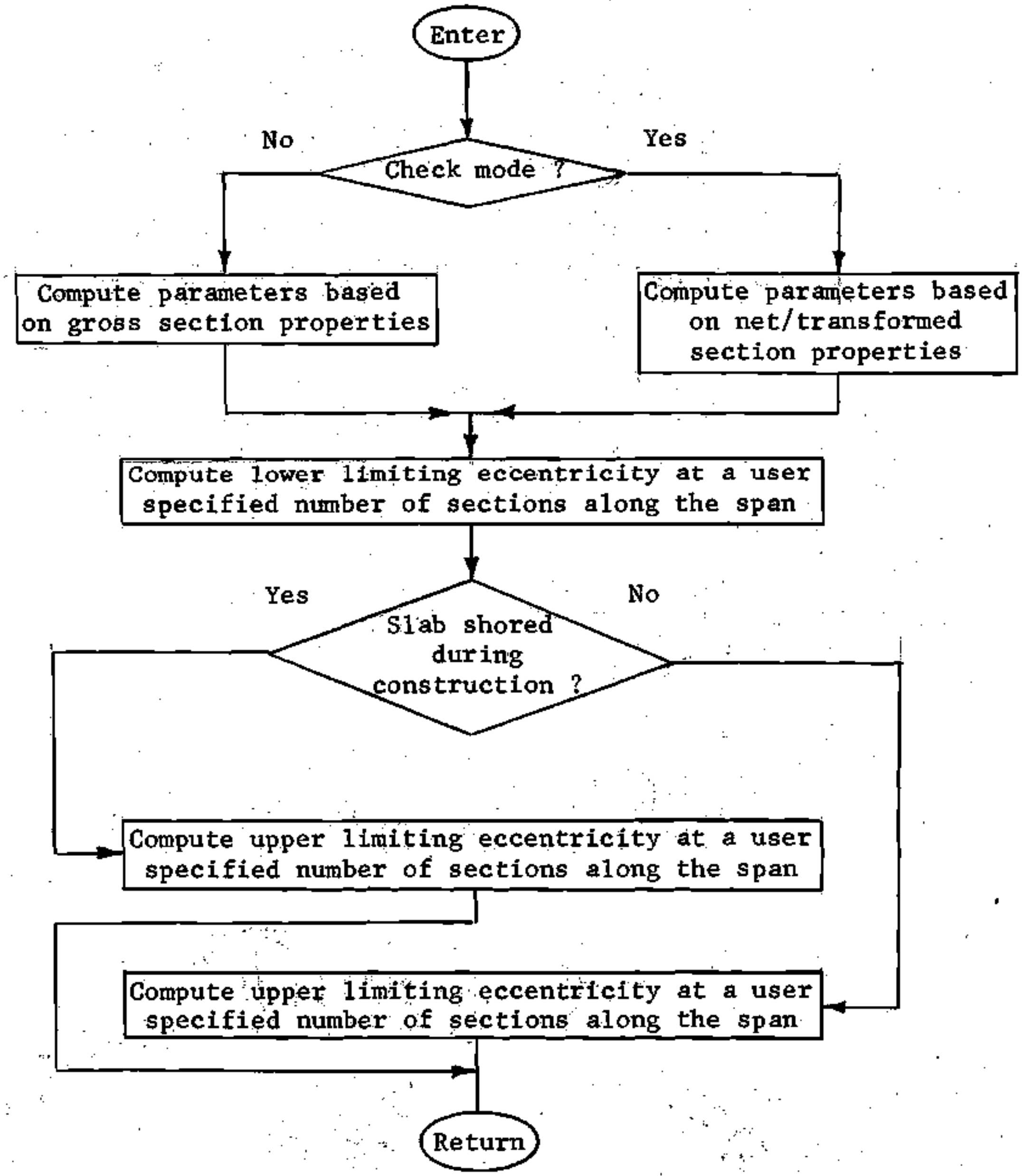
STRCX


Enter
Generate initial top and bottom stresses at a user specified number of sections along the span
Slab shored during construction ?
Yes
No
Generate final top and bottom stresses at a user specified number of sections along the span
Generate final top and bottom stresses at a user specified number of sections along the span
Return

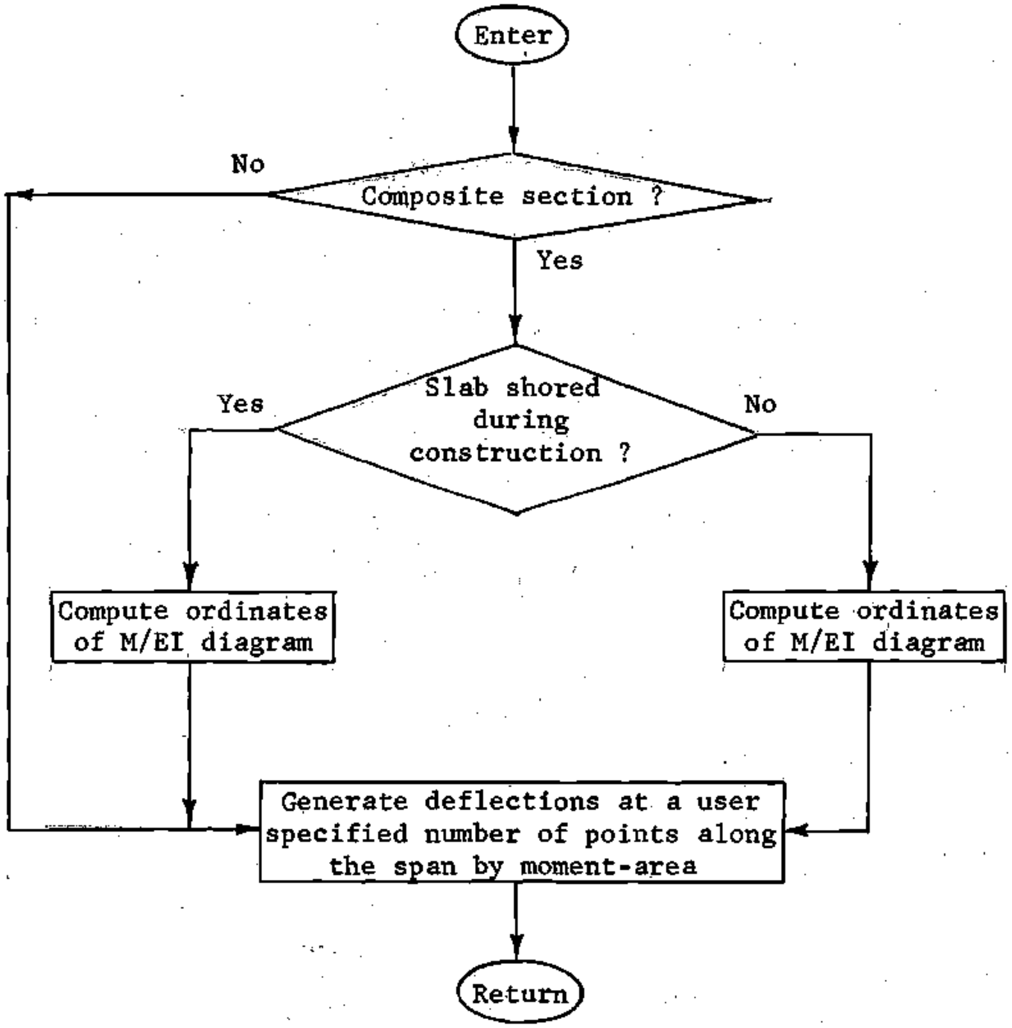
PROFIL


Enter
No
Check mode ?
Yes
Compute parameters based on gross section properties
Compute parameters based on net/transformed section properties
Compute upper and lower limiting eccentricities at user specified sections along the span
Return

PROFIC


Enter
No
Yes
Check mode ?
Compute parameters based on gross section properties
Compute parameters based on net/transformed section properties
Compute lower limiting eccentricity at a user specified number of sections along the span
Yes
No
Slab shored during construction ?
Compute upper limiting eccentricity at a user specified number of sections along the span
Compute upper limiting eccentricity at a user specified number of sections along the span
Return

DEFLN


Enter
No
Composite section ?
Yes
Yes
Slab shored during construction ?
No
Compute ordinates of M/EI diagram
Compute ordinates of M/EI diagram
Generate deflections at a user specified number of points along the span by moment-area
Return

DEFLTR

Enter

(Same as DEFLN except based on
net/transformed section properties)

APPENDIX C

FORMATTED INPUT

The purpose of this appendix is to present the exact format for all input information. It is assumed that the reader is, by now, familiar with the input parameters listed in Chapter IV and that he has a working knowledge of the FORTRAN IV language. We now define some additional parameters associated with the loading specifications.

Let

CN = combination number

NOL = number of loadings included under a given combination

LN = loading number

LTYPE = loading type, specify 1 for uniform load, 2 for uniform segment load, 3 for concentrated load, 4 for moving load

W = uniform or uniform segment loading intensity (kips/ft)

P = concentrated load intensity (kips)

A = distance in feet from left support to start of uniform segment load or to the line of action of a concentrated load

B = length in feet of uniform segment load

LD = moving load designation, specify 1 for H20-44, 2 for H15-44, 3 for H10-44, 4 for HS20-44, and 5 for HS15-44

LM = loading method, specify 1 for standard truck load,
2 for equivalent land load

DF = distribution factor

Input is read in by two basic methods. The problem specifications and section properties are input by namelists while the loading specifications are handled as standard formatted input. There are two namelists to be input: one which contains problem specification parameters (SPEC); and one which contains section dimension and section property parameters (PROP). The following is a list of FORTRAN statements defining the input:

```
    NAMELIST/SECT/ DES, TYPE, COMP, BOND, TENS, SUP, LEN, NSEG
    NAMELIST/PROP/ H, HMAX, HMIN, BBMAX, BBMIN, BBHMAX, BBHMIN,
   1BB, BWMAX, BWMIN, BW, MWHMIN, BT, BTMAX, BTMIN, BTHMAX, TB,
   2TBMAX, TBMIN, TBBMAX, TBBMIN, TTMAX, TTMIN, TT, TTTMAX, TTTMIN,
   3FCP, FCP1, FSP, FCP1, FTP1, FCP2, FTP2, TL, MEDA, GAMMA, YS,
   4CB, CT, EPCST, CRF, AS, FO, BS, TS, GS, ESLAB
    INTEGER  TYPE, BOND, DES, TENS, COMP, SUP, CN
    REAL     LEN, NEDA
 99 READ (5,99) R1, R2, R3, R4, R5, R6, R7, R8, R9, R10, R11, R12
    READ (5, SECT)
 98 READ (5, 100) CN, NOL
    IF (CN.EQ.0) GØ TØ 500
    LN = 0
    IF (NOL.EQ.0) GO TO 200
 50 LN = LN + 1
    READ (5,101) LTYPE
```

```
      GO TO (10, 20, 30, 40), LTYPE
   10 READ (5, 102) W
      TO TO 60
   20 READ (5, 103) W, A, B
      TO TO 60
   30 READ (4, 104) P, A
      GO TO 60
   40 READ (5, 105) LD, LM, DF
   60 IF (NOL.EQ.LN) GO TO 98
      GO TO 50
  500 READ (5, PROP)
      READ (5, 101) 1END
      IF (1END.EQ.0) GO TO 99
   99 FORMAT (12 A6)
  100 FORMAT (215)
  101 FORMAT (15)
  102 FORMAT (F10.0)
  103 FORMAT (3F10.0)
  104 FORMAT (2F10.0)
  105 FORMAT (215, F10.0)
   CALL EXIT
   END
```

Notice that the first card read is an alpha record. This is intended as a title card and the record punched in the first 72 columns is printed out at the beginning of the output. Also, it should be observed that the last card signals the end of the input record. If zero is specified, the program reads another complete set of data, but if one is specified, the program terminates. Any number of data sets may be included in a single run.

APPENDIX D

DEFAULT VALUES

In many instances, the program user is likely to leave some input parameters unspecified. If the parameter left unspecified is not essential to the formulating of the problem, a value will be assigned to that parameter based either on the AASHO specification requirements, the "normal" value usually associated with that parameter, or, as in the case of a dimension restraint parameter, a value which will allow an essentially unrestrained design. It should be noted that all parameters left unspecified are taken as zero. In some cases an assignment of zero may be a legitimate value; for these parameters, zero is the only default value assigned. The only parameters which must be assigned by the user are: TYPE, LEN, TENS, and NSEG. All others are either assigned default values or make sense as zero.

For example, if the user wished to design a prestressed, post-tensioned, I shaped highway bridge girder with a bonded, parabolic tendon and 50 foot span, he could specify only DES=1, TYPE=1, COMP=1, BOND=1, TENS=2, NSEG=10, and LEN=50.0, letting all other parameters assume default values. The resulting design will meet all AASHO specification requirements but may be impractical because certain dimensions are either too large or too small. Should the depth, for instance, be limited to a maximum of 30 inches, the user would also need to specify HMAX=30.0. Or if it were important that the flanges be thin compared to their widths, the user would specify TTTMAX=0.10 and TBBMAX=0.10. It is not necessary then, in many cases, to specify a great number of input parameters. Specifications limited to only a few parameters letting the others assume default values usually yield quite acceptable solutions.

Table 5. Default Values

Parameter	Default Value
	Problem Description Parameters
DES	Zero
COMP	Zero
TYPE	Must be specified either 1, 2, or 3
BOND	Zero
SUP	Zero
TENS	Must be specified either 1 or 2
LEN	Must be specified
NSEG	Must be specified as an even integer greater than 0 but less than 51
	Strengths and Stress Allowables
FCP	5000 psi
FCPI	FCP - 1000 psi
FSP	270,000 psi
FCP1	0.6 FCPI for pretensioned members* 0.55 FCPI for post-tensioned members*
FTP1	-3.0 FPCI *
FCP2	0.4 FCP*
FTP2	-3.0 FPC for pretensioned members (but not greater than -250 psi)* Zero for post-tensioned members*
	Tendon Description Parameters
TL	Zero
NEDA	0.89 for pretensioned girder 0.84 for post-tensioned girder
YS	0.1 H
AS	Zero
FO	Zero

*From Section 1.6.7 AASHO Standard Specifications for Highway Bridges (1965 ed.).

Table 5. Concluded

Parameter	Default Values
	Section Dimensions and Properties
HMAX	3.0 $0.1\ M_s$ *
HMIN	0.1 HMAX
BBMAX	1.5 HMAX
BBMIN	BWMIN for I and T sections 1.0 for rectangular sections
BBHMAX	2.0
BBHMIN	0.2
BWMAX	0.5 HMAX
BWMIN	3.0
BWHMIN	0.1
BTMAX	HMAX
BTMIN	BWMIN
BTHMAX	2.0
TBMAX	0.25 HMAX
TBMIN	3.0
TBBMIN	0.075
TBBMAX	0.75
TTMAX	0.25 HMAX
TTMIN	3.0
TTTMIN	0.075
TTTMAX	0.75
H	Integer value of HMAX
TB	0.2 H
TT	0.15 H
BW	0.15 H
CT	Zero
CB	Zero
EPCST	0.033 $\text{GAMMA}^{1.5}$ FCP
ESLAB	EPCST
BS	Zero
TS	Zero
GS	Zero
CRF	3.0
GAMMA	145 pcf

*From empirical equation presented by T. Y. Lin (8).

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